EFFECT OF RATE OF LOADING ON SETTLEMENTS

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CERTIFICATE

It is certified that this work 'Effect of Rate of Loading on Settlements' by S.G. Joag has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

July, 1982

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SYNOPSIS

Foundation design is one of the most important and challenging jobs in any civil engineering work. Foundations are designed from the considerations of bearing capacity and settlement. Behaviour of foundation is governed by the soil on which it rests. An experimental study was conducted in this thesis to assertain the effect of rate of loading on settlements.

Settlements are normally calculated at two conditions viz. short term and long term. In short term calculations no pore pressure dissipation is considered i.e. the effect of construction time is neglected. In the later case total dissipation of pore pressure is considered. This approach is highly conservative because the effect of construction time can not be neglected in all types of soils. The pore pressures do get dissipated during construction. In alluvial soils this dissipation is significant.

For this study, local Kanpur silt and clay were considered. Samples were prepared in C.B.R. moulds and were saturated. To simulate a foundation, a 40 mm diameter alluminium disc was used. Construction period, to build up a final stress level of 0.48 kg/cm^2, ranged from 1 hour to 16 hours. One set of experiments, in which loading period

was kept constant at 8 hours, but final stress level varied from 0.32 kg/cm^2 to 1.6 kg/cm^2, was conducted on silt.

It is observed that due to stress path effect (finite construction time) there is reduction in total settlement. Post construction settlement is only fraction of the total settlement. Slower the rate of loading, smaller settlements were observed. It was also observed that more than 85 percent settlement takes place during construction period itself. Slower the loading rate,less is the post construction settlement.

In appendix,a case study of rammed stone columns (Gravel pile) installed at two sites is made. Parametric study based on elastic continuum theory is carried out. It is observed that there is considerable increase in the stone column diameter due to ramming. Also the stone column capacity calculation from cavity expansion theory and pile formula give different results for the two sites. Due to limited data exact analysis could not be carried out.

CHAPTER 1

INTRODUCTION

Foundation Engineering plays an important role in design of any structure. The foundation of a structure is defined as that part of the structure in direct contact with the ground and which transmits the load of the structure to the ground. Direct bearing foundation or shallow foundations are constructed under the structures to transmit the loads to the soil. Hence the performance of a foundation depends on the soil on which it rests. When a soil is weak or compressible and is of considerable thickness, piled foundations are necessary to carry the load through such a weak layer to an underlying hard or stiff stratum.

Every foundation problem involves study of ultimate bearing capacity of soil under foundation load and settlement (consolidation settlement or differential settlement) under foundation load.

Ultimate bearing capacity is that value of net loading intensity at which the ground fails in shear. For calculations of ultimate bearing capacity the worst condition i.e. undrained condition ; when no pore pressure dissipation

is allowed, is considered. But certain silty soils like Kanpur silt are quite freely draining as compared to clayey soils. Hence the undrained condition does not always exist throughout the loading. There is some amount of drainage which changes the strength parameters of soil. Also the loading of the foundation is gradual i.e. construction type of loading. Considering that consolidation takes place during construction the bearing capacity might increase. The other extreme case could be a fully drained condition, which theoretically sppeaking would take infinite time .

Settlement prediction of foundations are always necessary from the view point of serviceability, utility and aesthetics of the structures. For the prediction of settlements, conventional theories do not take into consideration the dissipation of pore pressure and subsequent consolidation that takes place in soils during construction. Also due to gradual loading, sufficient time may elapse before next load increment, hence consolidation under the load takes place. This indicates that significant settlement might take place during construction period itself. It means that,if it is possible to predict the settlement that occurs during construction period, lower safety factor for post construction settlements can be considered. This in effect might lower the actual cost of foundation. Settlements are predicted for the worst and best condition

i.e. undrained and drained condition respectively considering construction time zero. But in reality it is first undrained loading followed by consolidation under the load.

Research was carried out to study the settlement behaviour of shallow foundation, when it is subjected to construction type of loading. The foundation was subjected to the same final load intensity but over different loading periods. Also over a fixed loading time different final load intensities were achieved.

In Chapter 2 a brief review of the work on bearing capacity of shallow foundations and settlement behaviour is given. Main emphasis is laid on foundation subjected to construction type of loading.

The details of the model tests conducted in the present study are presented in Chapter 3. Tests were conducted with a view to obtain the settlement behaviour of the footing under the construction type of loading. Soils used were Kanpur silt and a clayey soil. The test set up, soil properties and test details are described in Chapter 3.

Discussion of the results obtained from the model tests and conclusions there from are included in Chapter 4.

At the end of the thesis a relevent list of references referred for the study is given.

CHAPTER 2

REVIEW OF LITERATURE

2.1 Introduction:

In any foundation engineering design problem prediction of bearing capacity and settlement are important. Literature on prediction and estimation of bearing capacity and settlement is very extensive. Considerable work has been done based on soil properties, foundation dimensions etc.

2.2 Bearing Capacity of Soils:

It has been the aim of soil engineers to predict the bearing capacity of soils with due consideration to safety against failure, utility and economy and dependability. All the theories of bearing capacity of soil developed so far have a starting point from fundamental Prandtl's theory of indendation of rigid plastic, incompressible, weightless, continious, semiinfinite, homogeneous and isotropic medium. For the real soils having weight, cohesion, and angle of shearing resistance Terzaghi (1943) presented a widely accepted method.

The analysis of bearing capacity problem basically considers a feasible mode of shear failure of soil supporting

the foundation. By now three different modes of failure have been accepted viz.

i) General shear failure (Terzaghi 1943)

ii) Local shear failure (Terzaghi 1943, DeBeer and Vesic referred by Vesic (1975) and

iii) Punching shear failure (DeBeer and Vesic referred by Vesic (1975).

However no general numerical criterion exists for exact prediction of the mode of failure of soil loaded by a footing. Vesic (1973) defines a rigidity index I, as a rational parameter which will evaluate the relative compressibility and inturn the mode of failure.

All the methods of analysis of bearing capacity assume the Mohr-Coulomb and general shear failure criterion to hold good. Three standard and widely used techniques for the solution of bearing capacity problems are (i) slip line method (ii) limiting equilibrium method (iii) limit analysis. The details about these methods are available, Scott (1965), Harr (1966), Chen (1975), etc.

The slip line method (also known as the method of characteristics) has been used for solving bearing capacity problem by Krishnamurty (1972), Ko and Scott (1975) among others. Sokolovsky (1960) has given a method,using slip lines,to calculate bearing capacity for monotonously

increasing, decreasing or for that matter any type of load. ~~using slip lines~~. The limiting equilibrium method has been traditionally used for two-dimensional and axi-symmetric problems. The solutions involving three dimensional analysis are extremly complex. The method of slices,(Bishop 1953; Janbu 1957),is one of the very widely used methods of this class for solving bearing capacity problems. The limit analysis method is of relatively of recent origin. Application of limit analysis to the bearing capacity problem is presented by Chen and Davidson (1973).

2.2.1 Solutions to Special Problems:

The solutions for the bearing capacity of finite sized footings involve three dimensional analysis. Few solutions are presented for axi-symmetrical case of circular footing by Shield (1955), Cox et al. (1961) and Chen (1975). The simple and common practice of predicting the bearing capacity of foundations of finite size is by using correction factors based on empiricism and partly on the experimental basis. Terzaghi (1943) recommended correction factors for slope on a semi-emprical basis. Modified expressions are presented by De Beer (1970).

The effect of shearing resistance of the overburden soil has been studied by Meyerhof (1951) Skempton (1964),

Brinch Hansen,(referred by Vesic 1975) and approximate formulae suggested in the form of depth factors. However Vesic (1973) has cautioned not to use these depth factors for shallow foundations.

Meyerhof (1953), De Beer, Brinch Hansen, (Vesic 1975), Kezdi (1961), have given correction factors for the inclination and eccentricity of the foundation load. Effect of and correction factors for the influence of ground water table, roughness of foundation base, adjacent footing has been presented by Vesic (1973, 1975).

The bearing capacity solutions discussed so far are based on the assumption of incompressibility of soil and as such are applicable to general shear failure mode only. For other two modes of failure for compressible soils (local and punching shear failure) no rational method exists. As an approximation, Terzaghi's suggestion of using reduced values of cohesion and angle of shearing resistance is still considered valid. In this connection recent work of Vesic (1975) in the form of compressibility factors obtained from the cavity expansion theory (Vesic 1972) is noteworthy.

Bearing capacity of footings on slopes has been studied by Meyerhof (1953), Janbu (1957) , Reddy and Mogliah (1975). The basic equations of bearing capacity

factors have been modified by Meyerhof and Janbu.

Real soils in natural state exibit some non-homogeneity. Two basic kinds of nonhomogeneity are encountered in practice. The first kind is that of distinct soil layers of different strength and variable soil profile. The simplest situation of such non-homogeneous soil condition would be that of two layer profile in two characteristic conditions, (a) bearing stratum softer than the underlying stratum (b) bearing stratum stiffer than the underlying stratum. Button (1953), has analysed such situations for saturated clays in undrained condition for the first time. In contrast to this, the other type of non-homogeneity is increase or decrease of soil strength with depth. Solutions for such situations are presented by Kenny (1964), Raymond (1967), Basudhar (1976), Experimental study of bearing capacity in layered soil is reported by Brown and Meyerhof (1969). Tejchman (1977) has conducted model tests concerning bearing capacity of a strip foundation resting on stratified subsoil loaded with vertical and inclined forces acting axialy and eccentrically. Three principal problems were analysed:effect of stratified subsoil on foundation bearing capacity, effect of loading system on foundation bearing capacity and stratified subsoil effect

on settlement. Nakase (1981) has given a complete set of bearing capacity factors for rectangular footing on clays of undrained shear strength (ϕ=0) increasing linearly with depth. Siva Reddy (1981) have applied limit analysis for bearing capacity of strip footing on anisotropic and non-homogeneous clay by assuming a mechanism similar to Prandtl mechanism but with various wedge angles.

Ismeal and Vesic (1981) have carried out an experimental study to evaluate the effect of soil compressibility on ultimate bearing capacity of shallow foundations. Small scale model tests were performed on two frictional soils with identical shear strength characteristics, (C, ϕ), but different deformation characteristics (E, ν). Das (1981) has given an account of a limited number of laboratory model tests results for the ultimate bearing capacity of eccentrically loaded rough surface footing on sand layer with a rough rigid base at limited depth.

2.2.2 Time Effects:

The increase in bearing capacity resulting from consolidation of a normally consolidated soil under preload has been reported by Taylor and Ooi (1971). Stability is checked only at two times i.e. before the start of the process of consolidation and after completion of the consolidation process. The construction period is

neglected and $\phi = 0$ analysis used in investigation.

The effect of construction time is very significant in case of alluvial soils where dissipation of excess pore pressure takes place simultaneously with construction activity. Under such circumstances use of conventional undrained analysis for stability at the end of construction is very conservative. Lumb (1965), gives a guideline that drained strengths can be used for all foundations on silty sand but that the undrained strength should be used for all foundations on clays.

Increase in bearing capacity with time because of dissipation of excess pore pressure is studied by Madhav and Vitkar (1978). Correction factors for construction time are given.

2.3 Settlement of Foundation :

Prediction of settlement forms the other important aspect of foundation behaviour. Total settlement comprises of 3 components viz. (i) immediate or undrained settlement which occurs immediately on application of the load, (ii) consolidation settlement, which occurs primarily because of the dissipation of excess pore pressures in the soil and is time dependent and (iii) creep settlement(secondary consolidation).

An attempt to account for the three-dimensional nature of settlement process has been made by Skempton and Bjerrum (1957). However their approach is semiempirical. An analysis of relative importance of immediate settlement is made by Davis and Poulos (1968), for a circular footing resting on the surface of ideal elastic soil. Prediction of initial settlement of structures on clay is made by D'Appolonia, et al. (1971).

2.3.1 Methods of Settlement Analysis:

(i) Stress path method:

Results of triaxial testing have been the basis for the stress path method. The main limitation of this method is in choosing the representative soil elements to test in a rather complex situation. In this method the most important step is to duplicate the initial stresses and total stress changes in laboratory tests. The stress path method gives larger strains near ground surface but gives smaller strains at depth. Studies by Simon and Som (1969), present the influence of change in lateral stresses on the deformation of London clay.

(ii) Generalised elastic method:

A general solution for immediate and final settlement using elastic theory was put forth by Davis and Poulos (1968). Triaxial and consolidation tests are conducted to

determine the elastic constants of soil. Poulos has cautioned that the successful application of this method depends on the choice of appropriate values of soil deformation moduli E_u, ν_u, E' and ν'. A similar approach has been used by Egorov et al. (1957) and they have compared the calculated and observed settlements of building at Moscow University. It has been reported that the deviation between the calculated and observed values of settlements is very slight. Elastic method is useful when the settlements must be known at many points other than just at the center line alone.

(iii) Finite element method:

With the fast development of the finite element methodology and availability of high speed computers, many difficulties encountered in earlier methods have been overcome. Recently Desai and Abel (1972) and Desai and Christian (1977), have presented use of finite element method to various geotechnical engineering problems. With this method the previously unsolved problems have been tackled.

(iv) The cambridge approach:

Extensive research has been carried out at Cambridge University into stress-strain behaviour of clays, Burland (1969, 1971). Normally and lightly overconsolidated

clay is treated as a work hardning plastic material. The soil is assumed to satisfy the normality condition and possess yield locus from which volumetric strain and shear strain increments are obtained. From these strain increments,vertical strain increment and finally the settlement is predicted.

(iv) Empirical methods:

Empirical approaches, based on large number of case studies may be used to supplement theoretical analyses or for crude preliminary estimates. The two most widely used empirical, or semiempirical methods are the load test and the penetration test.

(a) Load Test:

In the load test,the soil is subjected to a load increase in stages with settlement under each increment of load being measured. The measured load-settlement data are then used to predict the behaviour of the actual footing. A widely used relation between settlement on sand and footing size is empirical one developed by Terzaghi and Peck (1967):

$$\frac{\rho}{\rho_o} = \frac{4}{(1 + D_o/D)^2}$$

where ρ = settlement of prototype,

ρ_o = settlement of test footing,

D = the smallest dimension of prototype,

D_o = the smallest dimension of test footing.

Indian standard code (IS 1888) gives following relation:

For sandy soils

$$\text{settlement of prototype} = \left[\frac{B_1\ (B_2 + 30.48)}{B_2\ (B_1 + 30.48)}\right]^2 \rho_o$$

For clayey soils

$$\text{settlement of prototype} = \frac{B_1}{B_2} \times \rho_o$$

where B_2 = width of test plate in cms,

B_1 = width of footing in cms.

To get dependable results from load test one must be sure that the soil at the site is homogeneous for a depth which is large relative to the size of the actual footing.

(b) Penetration Test:

Various penetration tests such as standard penetration tests, Dutch deep sounding tests have been used to predict settlement of foundation on sand. The most widely used in the standard penetration test. Relation proposed by Meyerhof (1965) gives,

$$\Delta q_s = N \rho / 8 \qquad B \leq 4 \text{ ft.}$$

$$\Delta q_s = \frac{N \rho}{12} \left(\frac{B+1}{B}\right)^2 \quad B > 4 \text{ ft.}$$

where Δq_s in tons/ft.^{2}, is the surface stress required to cause a settlement of one inch for a footing resting on sand. B is in ft. and ρ is in inch.

2.3.2 Theories of Consolidation:

A simple consolidation theory is presented by Terzaghi (1943). Main limitation of the Terzaghi's theory is that it is valid for one dimensional conditions only. It has been recognised that in most field cases, consolidation may not be one-dimensional. Terzaghi's theory has been modified for three-dimensional flow conditions by Rendulic. Terzaghi-Rendulic theory is also called as a pseudo three-dimensional theory. A complete theory of three-dimensional consolidation for an ideal soil is presented by Biot (1941). Biot's theory satisfies in addition to continuity, the requirement of displacement compatibility. Gibson and Lumb (1953), have presented very useful solution of consolidation problems by numerical technique using finite differences. Extensive use of Terzaghi-Rendulic theory has been made by Davis and Poulos (1968, 1972).

The theory of consolidation is used to predict the rate of settlement. Exact degree of settlement can be predicted by Biot's theory which is applicable to ideal

soil. Using Terzaghi-Redulic theory an important contribution has been presented by Davis and Poulos (1968). The authors have shown that the degree of settlement is approximately equal to the degree of pore pressure dissipation on a selected vertical line, by adjusting the time factor scale in U_p vs T relationship. Some solutions for the rate of settlement of strip and circular footings subjected to instantaneous loading are presented by Davis and Poulos.

Anisotropy and variations of certain coefficients with depth have been subject matter of some research workers. Schiffman and Gibson (1964) have developed the solutions for non-homogeneous clay layers in which K and m_v are considered varying with depth. Another theory which takes into account variation of K and m_v with time is presented by Davis and Raymond (1965). Later this nonlinear theory is modified by Davis (1971), to take into account the effect of layer depth.

Foott and Ladd (1981) have presented an approximate method for prediction of immediate settlement based on settlement ratio. It takes into account soil modulus E_u and influence factor I_ρ. Soneja and Jain (1981) have studied the effect of footing width on settlements in silt of low compressibility. Based on field and laboratory tests, equations have been developed, correcting the settlement.

An important aspect is time dependant loading and its effect on degree of settlement. Solutions for such case for one-dimensional analysis is given by Schiffman (1958). Studies on consolidation under construction loading with varying c_v are made by Madhav (1967), however, this study is valid for one-dimensional case only.

In spite of many advances and researches in the settlement theories, no solution exists for the problem of construction time dependent loading for two and three-dimensional conditions. In alluvial soils dissipation of excess pore pressure is relatively fast and significant consolidation may take place during the period of construction itself. Madhav and Vitkar (1981) have given a method to predict settlement of strip and circular footings subjected to construction type of loading. A finite difference scheme is used for the Terzaghi-Rendulic theory of consolidation for plane strain and axisymmetric cases. Different drainage boundary conditions and the effect of anisotropy with respect to permeability are also considered.

Adachi and Todo (1979) have concluded that rate of settlement (consolidation) in field was much faster than the rate estimated by using the coefficient of consolidation obtained from laboratory test. The water content was as high as 80-110 percent. About 50 percent settlement took place during construction period itself. For silty sand

Tominaga et al. (1979) have observed that about 60 percent settlement occurs during construction period. In papers published by Kotzias and Stomptopoulous (1969), Schnabel (1972), Kezdi et al. (1976) etc. it can be observed that in most cases construction time settlement is more that 65 percent.

D'Appolonia et al. (1971) state that the initial settlement that takes place due to undrained shear may constitute a large portion of total final settlement depending upon the nature of soil, the loading geometry, the thickness of compressible layer etc.

Present study is an experimental verification of Madhav and Vitkar (1981) settlement analysis.

CHAPTER 3

MODEL STUDIES

3.1 Introduction:

One of the most common procedures for finding out the settlement behaviour of a shallow foundation when it is subjected to construction type of loading is to conduct actual field tests and corelate the information thus obtained to predict the behaviour of other foundations. But such a procedure will not be always cost effective. Some preliminary estimate has to be made.

Major constraint in laboratory testing is the simulation of actual field condition i.e. stress field, nonhomogeneity etc. Model tests conducted in the laboratory may be helpful to arrive at the nature of the load-settlement relation.

A series of model tests were conducted on remoulded clay and remoulded local Kanpur silt and to simulate the foundation, a circular alluminium plate 40 mm in diameter and 6 mm thick was used.

3.2 Soils and Test Details:

3.2.1 Soils:

Soils that were considered were local Kanpur silt and clay. Tests were conducted to obtain liquid limit and

plastic limit, O.M.C. , and maximum dry unit weight.

Kanpur Silt:

Liquid limit	32 percent
Plastic limit	17 percent
Plasticity index	15
Optimum moisture content	17 percent
Maximum dry unit weight	1.75 gm/cc.

Clay:

Plastic limit	15 percent
Liquid limit	34 percent
Plasticity index	19
Optimum moisture content	16 percent
Maximum dry unit weight	2.05 gm/cc.

From the plasticity chart Kanpur silt and clay fall into CL type of soil.

Oedometer tests were conducted to obtain the consolidation characteristics of the two soils.

Kanpur silt	C_c	=	0.1446
	c_v	=	0.1814 cm^2/min.
Clay	C_c	=	0.1962
	c_v	=	0.126 cm^2/min.

3.2.2 Preparation of Samples:

Soil was either oven dried for 24 hrs or sundried

sieve IS 80 (850 microns). The idea being to remove the coarser particles. For compaction static compaction was used.

By static compaction, the desired dry density and moisture content can be obtained directly. The weight of wet soil at the required moisture content to give the intended dry density, when occupying the standard specimen volume is calculated as follows,

Soil was compacted in standard C.B.R. mould of size 15 cm dia and 17.5 cm height.

$$\text{Volume of C.B.R. mould} = \frac{\pi}{4} \times 15^2 \times 17.5$$

$$= 3092.51 \text{ cm}^3$$

If intended dry density is γ_d gm/cc., the weight of dry soil in C.B.R. mould $= 3092.51\ \gamma_d$ gm

$$= 3.09251\ \gamma_d \text{ kg.}$$

If the intended moisture content is w percent the weight of wet soil is C.B.R. mould

$$= \frac{100+w}{100} \times 3092.51\ \gamma_d \text{ gm}$$

$$= \frac{100+w}{100} \times 3.09251\ \gamma_d \text{ kg.}$$

For Kanpur silt γ_d $= 1.75$ gm/cc , $w = 17$ percent

Weight of dry soil $= 1.75 \times 3.09251 = 5.41$ kg.

Weight of wet soil $= 1.17 \times 5.41 = 6.33$ kg.

Similarly for clay $\gamma_d = 2.05$ gm/cc. , $w = 16$ percent

Weight of dry soil = 6.34 kg.

Weight of wet soil = 7.35 kg.

A batch of dry soil was weighed and mixed with water to obtain a required moisture content. Correct weight of wet soil was placed in the mould and compacted by pressing in a 2 inch thick displacer disc, a filter paper being placed between the disc and the soil. When the top of the displacer disc is flush with the rim of the mould, the required volume of specimen was obtained, although with some soil types it may be necessary to continue loading untill displacer disc is just below the ring, in order to allow for elastic recovery when the load is released.

After compacting by static method, the mould along with the collar was placed in water for saturation, with a spacer at the bottom of mould. The top of the soil is covered with filter paper and a perforated disc. To hasten the saturation process it was decided to put filter paper all round the inside periphery of the mould before putting the loose wet soil. This hastened the process because radial permeability is greater than either horizontal or vertical permeability. The mould was put in water for at least for 5 days. After eight days of saturation the average water content along the height of the mould for the two soil was:

Clay	Top	24 percent
	Middle	24.18 percent
	Bottom	26.63 percent
Kanpur silt	Top	28.02 percent
	Middle	28.16 percent
	Bottom	29.15 percent

After a minimum of 5 days of saturation, the mould was taken out of water and was allowed to drain ... . Coller and perforated plate were removed, alluminium disc was placed at the center of the soil. On top of the disc a plunger of 20 mm diameter and 50mm thickness was put to facilitate the load transfer from the loading frame to the foundation. To prevent loss of water due to evaporation, cotton waste was put around the aluminium disc, to cover the top of the soil. This cotton waste was kept saturated by pouring water on it during the experiment.

3.2.3 Loading Arrangements:

For loading, standard consolidation frame with certain modification was used. The frame height was increased to accomodate C.B.R. mould. The lever ratio of the frame was 1:10.

The main disadvantages with consolidation frame are, the load placement gives a jerk to the specimen and the loading is in steps (nonuniform for consolidation test).

Hence by the usual arrangement construction type of loading, cannot be simulated. Hence a modification of the loading arm was made.

The load arm on which loads are hung was removed, instead a bucket (4 litter capacity) with 2.2 kg. weight in it was hung.

For loading, lead or iron shots could have been used, but water has an advantage over others that 1 ml of water weighs 1 gm. Also during unloading removing of shots from bucket would have posed a problem, hence it was decided to use water for loading. 50 ml capacity burette was attached to the frame. Control of loading rate was easier with water as compared to lead or iron shots. Loading rate was adjusted by adjusting the burette tap. For unloading water was sucked out with a 25 ml capacity pipette. To check evaporation losses from the bucket a thin film of oil was maintained on top of water surface. While loading it was carefuly checked that the water does not hit the bucket as it drops from the burette. The water was allowed to glide along the surface of the bucket by adjusting the position and the height of the burette.

3.2.4 Loading Details:

A series of experiments were conducted to obtain a safe final load value for both the soils. It was observed

that 600 ml of water when poured, the loading is quite safe. Beyond this the load arm starts tilting. Minor adjustments to the arm do not create much difference, but major adjustment, to make the arm horizontal, pushes the alluminium disc into the soil.

600 ml of water $=0.6$ kg.

Area of disc $= \pi/4 \times 4^2 = 4\pi \text{ cm}^2$

Load intensity $= 0.6/4\pi = 0.0477 \text{ kg/cm}^2 = 0.048 \text{ kg/cm}^2$

Lever ratio 1 : 10.

Final load intensity on the foundation = 0.48 kg/cm^2.

On Kanpur silt two sets of experiments were conducted (1) same load intensity but different loading period, (2) same loading period but different load intensities.

In the first set 0.48 kg/cm^2 was applied in 1 hr., 2 hrs., 4 hrs., 8 hrs. and 16 hrs. Readings at an interval of 10 min. Unloading was also done in same steps.

In the second set, the loading period was kept constant at 8 hours, but load intensities were varied. Thus loading rate was 0.04 kg/cm^2/hour, 0.06 kg/cm^2/hour. 0.08 kg/cm^2/hour, 0.1 kg/cm^2/hour, 0.12 kg/cm^2/hour, 0.16 kg/cm^2/hour and 0.2 kg/cm^2/hour. Readings at an interval of 10 min. Unloading was done in same steps.

On clay only one set of experiments were conducted. Load intensity was kept constant at 0.48 kg/cm^2 , but loading

period was varied. Loading period was 1 hour, 2 hours, 4 hours, 8 hours and 16 hours. Readings at an interval of 10 min. were taken. Unloading was done with same rate.

Before unloading, for both Kanpur silt and clay, final load was kept constant till the settlement was stabilised i.e. settlement rate w.r.t. time was constant and close to 0.01 mm/hour or less.

3.3 Other Tests:

After the experiment, thin walled 1.5 inch dia tube was pushed into the mould and samples for triaxial test were obtained. Two types of triaxial tests were conducted on both soils: CD (consolidated drained) and UU (unconsolidated undrained or quick test) to obtain the stress-strain characteristics. Cell pressures were kept to 1 kg/cm^2, 2 kg/cm^2 and 3 kg/cm^2.

To obtain consolidation characteristics oedometer test was conducted on both the soils.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Results:

In this chapter the results of the various tests conducted are presented. The load-settlement and time-settlement behaviour of the foundation on the two soils viz. Kanpur silt and clay is presented catagoricaly with respect to the soil. Also the discussion and conclusions from the results is presented.

Figure 1, depicts a typical construction type of loading with a construction period of 't_o'. In Figure 2, the experimental set up is shown. Oedometer test results for the two soils are presented in Figures 3 and 4 and can be summarised as:

Soil	C_v cm^2/min.	C_c
Kanpur silt	0.1814	0.1446
Clay	0.1260	0.1962

UU or undrained test, and CD or drained test results for the two soils under three confining pressures of 1 kg/cm^2, 2 kg/cm^2 and 2.9 kg/cm^2 are presented in Figures 5 and 6.

(a) Kanpur Silt:

Experiments were conducted to obtain the load-settlement behaviour under two conditions.

I) Load intensity constant at 0.48 kg/cm^2.,but loading period varying from 1 to 16 hrs.

II) Load intensity varying from 0.32 kg/cm^2 to 1.6 kg/cm^2., but loading period constant at 8 hours.

Time-settlement, load-settlement and log-time settlement curves for the first set are presented in Figures 7 to 9. Figures 10 and 11 give time-settlement and log-time settlement curves for second set of experiments.

(b) Clay:

On clay specimen, the load intensity was kept constant at 0.48 kg/cm^2, but the loading period was varied from 1 to 16 hours. Figures 12 through 14 present the time-settlement, load-settlement, log time-settlement curves.

Figure 15 shows log time-settlement plot for clay and silt. Settlements are plotted for t_o and t_f. $\rho t_o/\rho_{final}$ versus t_o plot for both the soils is presented in Figures 16. Fig. 17 depicts curves for final settlement versus t_o for the two soils. Figure 18 presents plot of rate of loading versus settlement at t_o for the second set of experiments on Kanpur silt.

4.2 Discussion:

It can be observed from Figure 7 that for Kanpur silt, when the loading period from 1 hr. to 16 hrs, the final settlement varied from 3.4 mm to 5.0 mm. Settlement at the end of loading period varied from 3.2 to 4.4 mm. The maximum settlement occured under the fastest rate of loading used i.e. 0.48 kg/cm^2/hour. On the other hand the slower the rate of loading, smaller have been the settlement . The ratio, $\rho_{rebound}/\rho_{final}$ varied from 0.06 to 0.09, with an average of 0.08.

When the rate of loading on Kanpur silt was varied from 0.32 kg/cm^2 to 1.6 kg/cm^2, but construction time was kept constant at 8 hours, the final settlement varied from 4.5mm to 21.0 mm. This can be concluded from Figure 10. Slower the rate of loading, less settlement was observed.

Figure 12, shows that, for clay the final settlement varied from 0.7 mm to 3.5 mm,when the loading period was varied from one hour to sixteen hours, while the load intensity was kept constant at 0.48 kg/cm^2. The maximum settlement occured under the fastest rate of loading i.e. 0.48 kg/cm^2/hour. Settlement at the end of construction varied from 0.6 to 2.8 mm. The ratio of $\rho_{rebound}/\rho_{final}$ was in the range of 0.04 to 0.1 with an average of 0.07.

The effect of the rate of loading on settlement is significant, as can be seen in case of clays. The variation of the final settlement is about 5 times in clays, as compared to about 1.5 times in case of silt. This is because silt is more permeable than clay, more dissipation of pore pressure takes place during construction period, hence less post construction settlement. In case of clays the settlement at the end of construction is less but post construction settlement is more as compared to silt.

From Figure 16, it can be observed that the percent settlement during construction period i.e. ρ_{t_o}/ρ_{final} increases with loading period. For Kanpur silt, it varied from 87 percent to 95 percent, with heigher percentage for slower loading rate. Same is true for clay with a range of 86 percent to 94 percent.

Figure 18 depicts a 'S' type curve. A rapid change in slope is observed after a 'critical loading rate'. This critical rate can be obtained by drawing tangents to the curve as shown in Figure 18. The point where the two tangents meet is the 'critical loading rate'. It can be observed from Figure 10 that there is marked increase in settlement beyond the critical loading rate of 0.16 kg/cm^2/hour.

4.3 Specimen Settlement Computations:

The vertical stress on an element at a depth d/2 below foundation will be 0.6465 x 0.48 = 0.31 kg/cm^2.

d is the diameter of the foundation = 4 cm

0.6465 is the influence factor from Boussinesq's equation. Strains at this stress level i.e. 0.31 kg/cm^2 are obtained from undrained and drained tests on Kanpur silt (Fig. 5). Undrained and drained settlements from these strains are also obtained.

Cell pressure kg/cm^2	Strains in UU Test Percent	Strains in CD test Percent	Undrained settlement mm	Drained settlement mm
1.0	1.50	0.15	2.625	0.262
2.0	0.75	0.10	1.313	0.175
2.9	0.40	0.05	0.700	0.088

Undrained settlement varies in the range of 0.7 to 2.6 mm. It is observed that smaller the confining pressure, the more is the settlement. Hence, if a triaxial test is conducted closer to a confining stress developed due to loading, in the laboratory specimen i.e. 0.064 kg/cm^2, then the actual settlements and computed settlements would have been comparable.

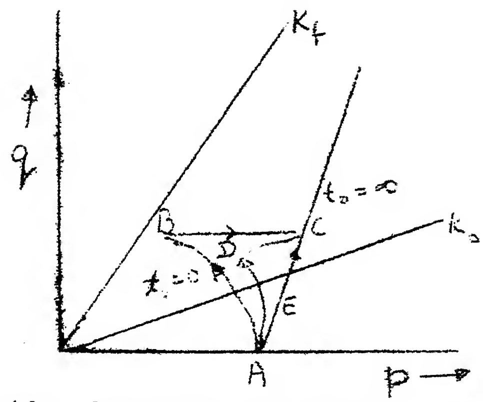
Consolidation settlement, for Kanpur silt for the stress level of 0.31 kg/cm^2 of an element at d/2 level works out to 36.66 mm. The consolidation settlement is very high because the original stress at the level is very less as compared to increase in stress i.e. $P_o << \Delta P$. But in field, it won't be so. The consolidation settlement and undrained settlements would be of the same order of magnitude.

Similar calculations for clay show,that the undrained settlement varies from 1.2 to 12.25 mm and consolidation settlement works out to about 53 mm.

For the second set of experiments on Kanpur silt, settlements are computed for two extreme load intensities of 0.32 kg/cm^2 and 1.6 kg/cm^2. For the load intensity of 0.32 kg/cm^2, undrained settlements varies from 0.175 to 8.75 mm, drained settlement in the range of 0.035 mm to 0.175 mm. Consolidation settlement is 31.34 mm. For the load intensity of 1.6 kg/cm^2, undrained settlement is very high. This is because failure has taken place at much lower stress level, (except for cell pressure of 3 kg/cm^2). Drained settlement varied from 0.525 to 1.75 mm. Consolidation settlement was 63.30 mm.

4.4 Conclusion:

The theoretical loading path for the type of loading considered in the study, would be ABC. A to B is undrained



loading till the desired intensity is achieved and from there on the soil undergoes consolidation (B to C) under that loading. The settlements are calculated for two extremes viz. undrained loading i.e. path AB and fully drained i.e. path AC. As can be seen from the figure, both conditions estimate wrong settlements. Even if settlements are estimated by considering path ABC, the settlements would be much more because in most soils there is a certain amount of drainage during loading, due to time-dependant nature of the loading considered. Hence the actual loading path will be ADC. Larger the loading period, the path moves towards AC. AC path implies an infinite construction time on the other hand less the loading time, the path will shift towards AB . AB path implies construction time of zero.

Hence it is highly unrealistic to estimate the settlements based on either fully drained or undrained condition. This is particularly true for alluvial soils. Path ABC will overestimate settlements, because it assumes

that construction time is zero hence no dissipation of pore pressure during loading. Hence any estimation of settlements for the construction type of loading must take into account the dissipation of pore pressures during construction period. This will reduce the magnitude of post construction settlement and the total settlement. This fact is brought in Figure 16. The post-construction settlements for the clay are more than that for silt, as the silt is more permeable than clay even though the nature of curves for both the soils is the same.

The laboratory study presented in this thesis brings out the importance of construction time and the rate of construction loading in predicting the settlement under field situation, where significant amount of excess pore pressure dissipates during the construction period.

It is confirmed that the conventional approach of not considering the dissipation of pore pressure during construction period is highly conservative. Hence for the design of foundation from settlement point of view, settlement occuring during construction period should be considered as it forms a major portion of total settlement. Also due to the particular nature of alluvial soils in which fully undrained condition is not possible, total settlements will be less. Longer the construction period more will be the settlement during construction period. If this fact is

considered, factor of safety for post construction settlement and total settlement can be reduced, thus lowering the actual construction cost.

An attempt to compare the experimental results with Madhav and Vitkar (1981) (Fig. 19) showed some discrepancy in that the experimental results (H/B=4.25) lie much above the theoretical curves possibly because the former include undrained deformations while the latter do not.

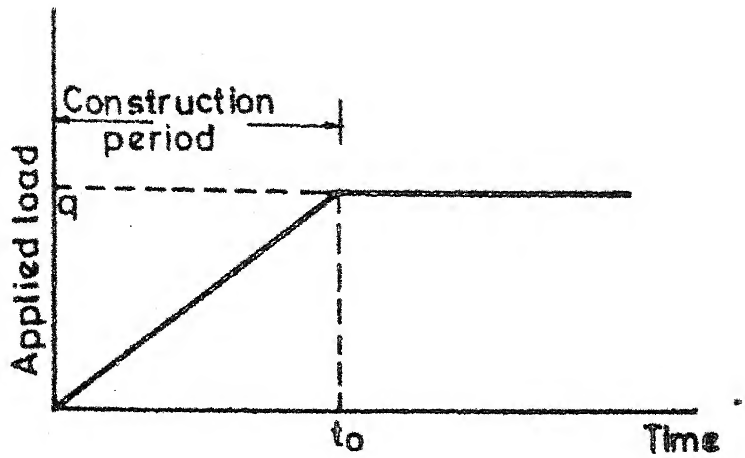


FIG.1 TYPICAL CONSTRUCTION TYPE OF LOADING.

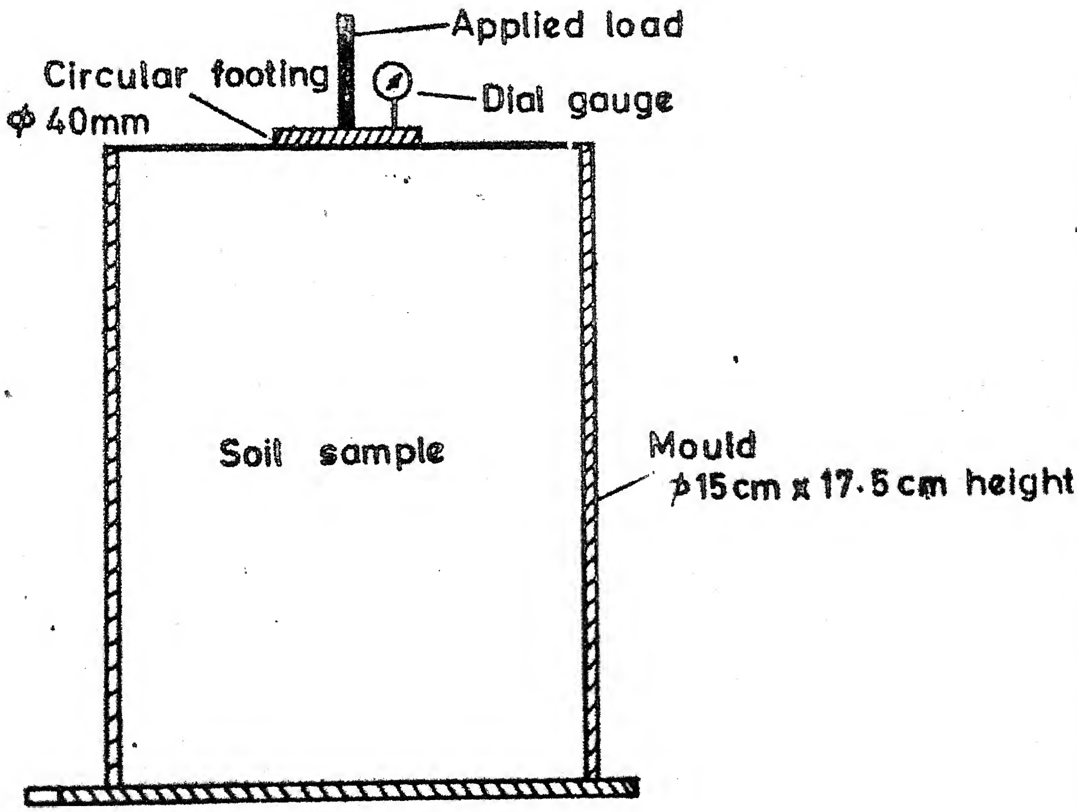


FIG. 2 EXPERIMENTAL SETUP

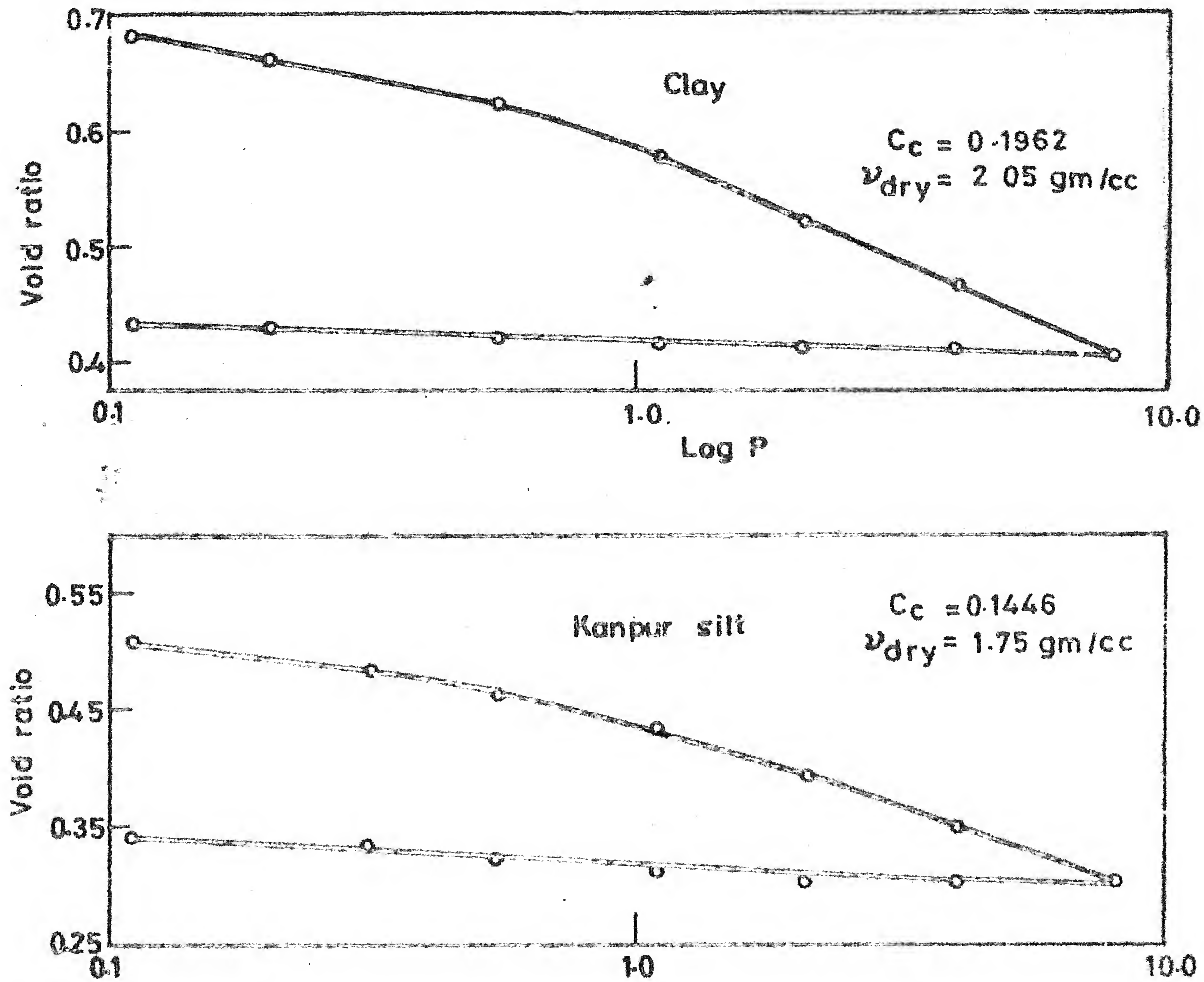


FIG.3 e - LOG P CURVES FOR TWO SOILS

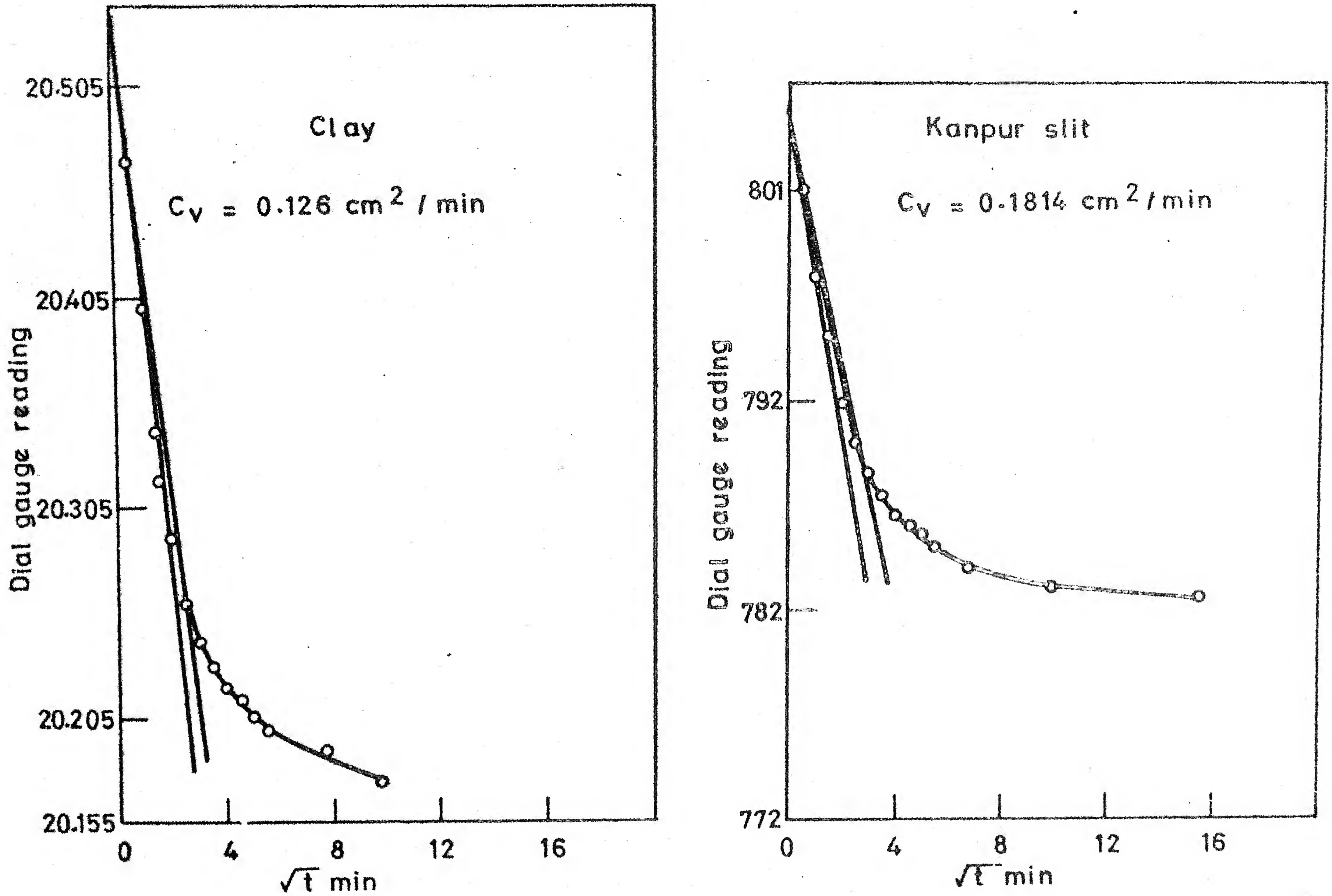


FIG.4 OEDOMETER TEST DATA FOR TWO SOILS

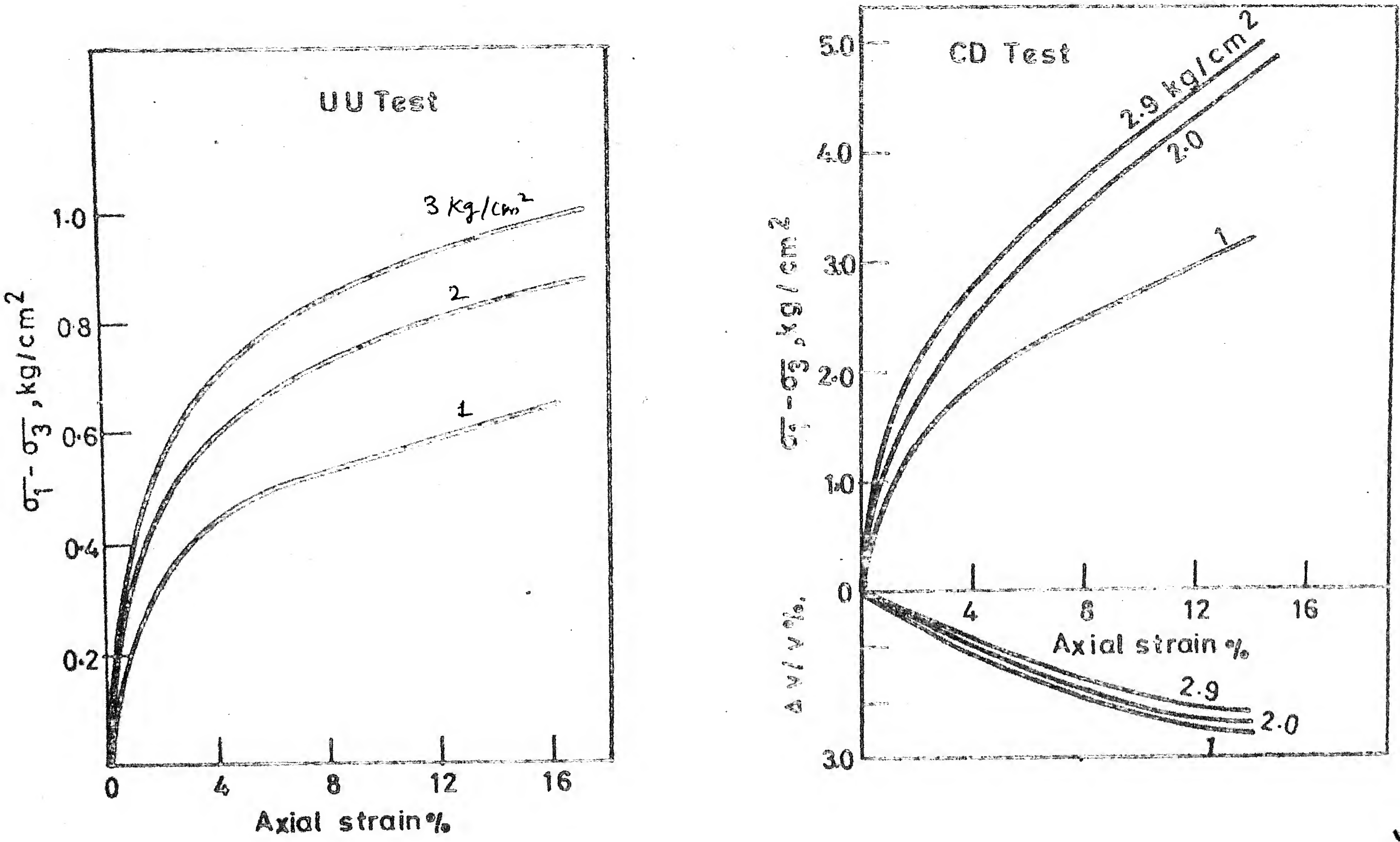


FIG. 5 TRIAXIAL TEST RESULTS FOR KANPUR SILT

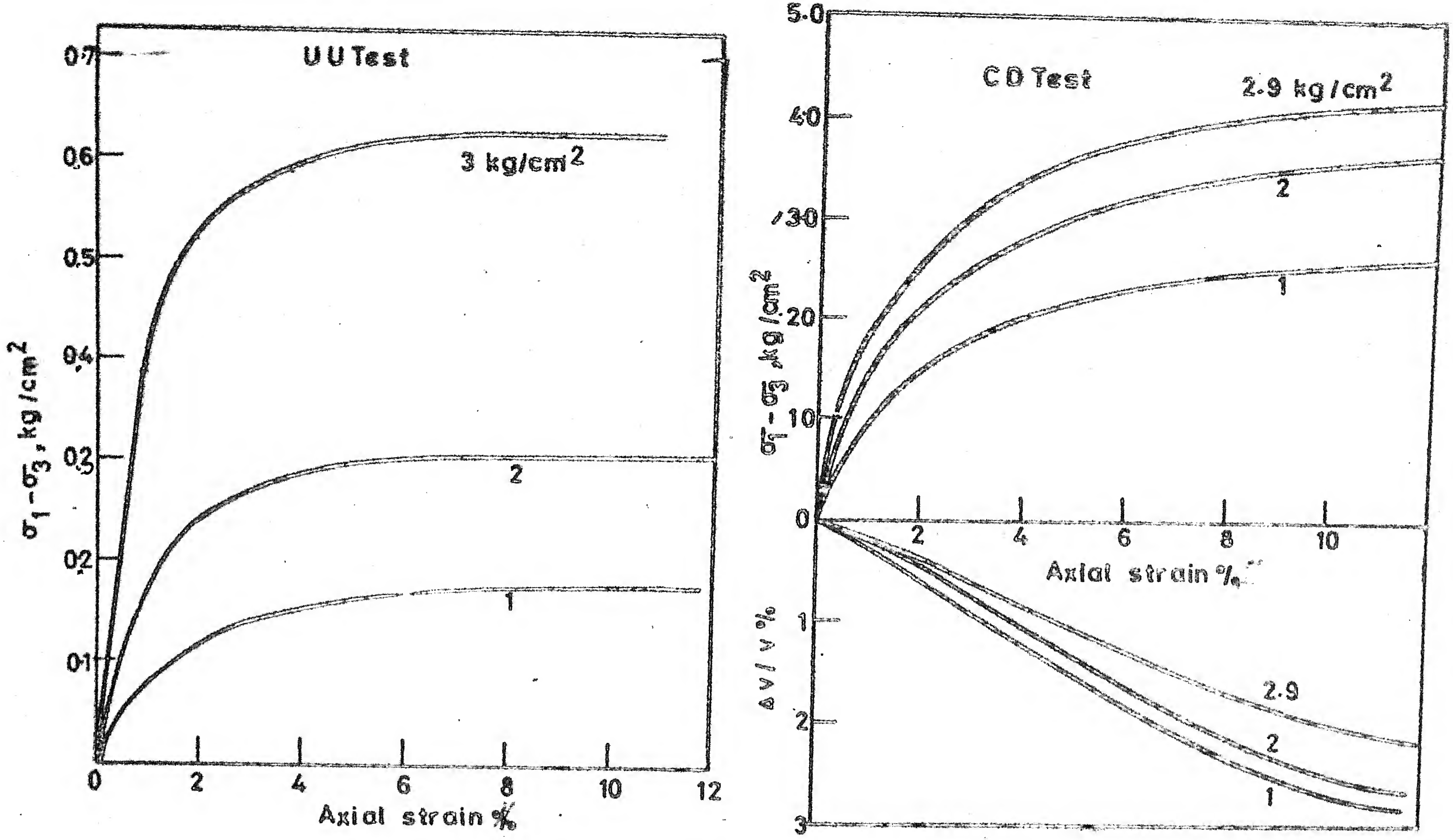


FIG. 6 TRIAXIAL TEST RESULTS FOR CLAY

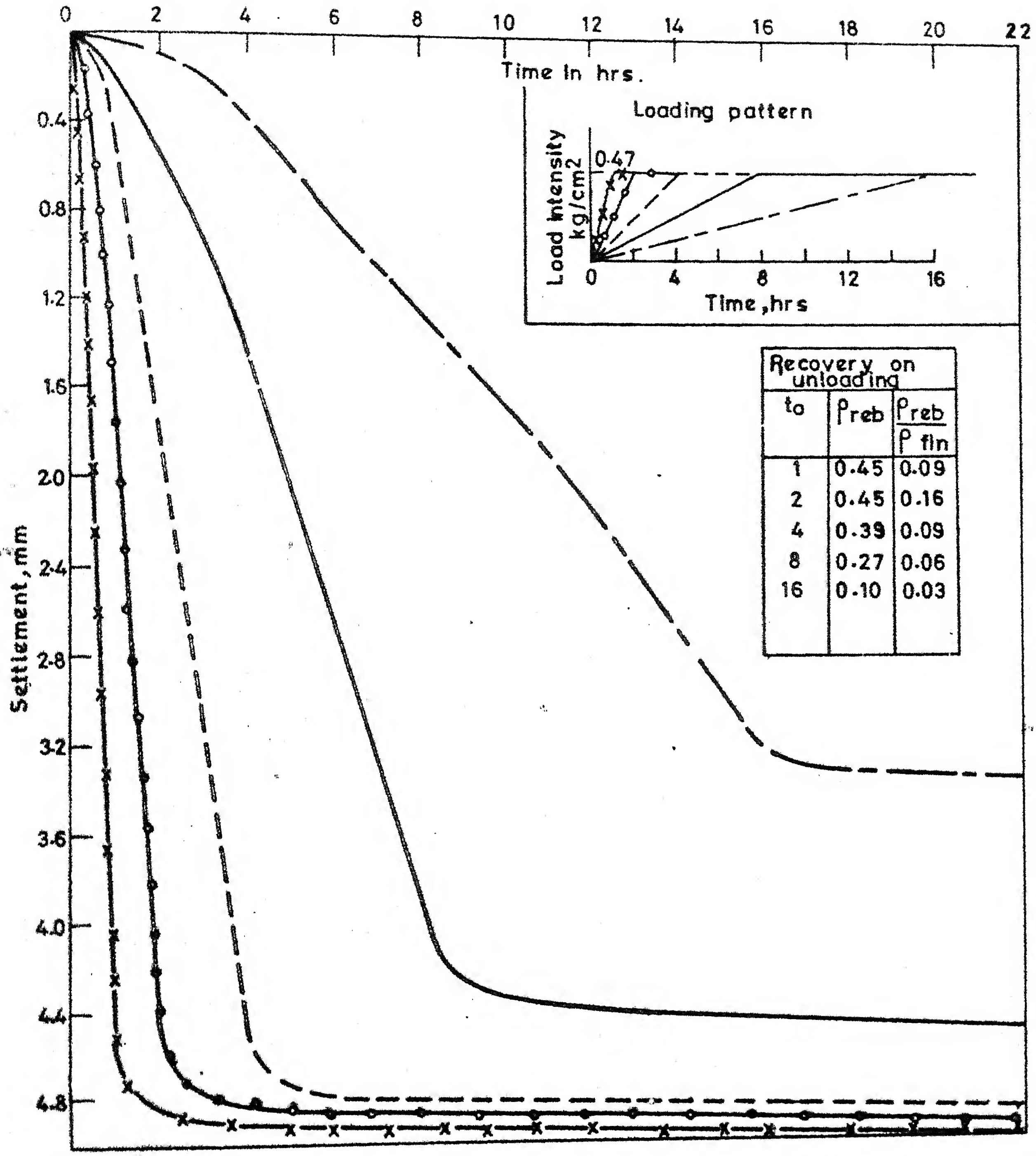


FIG.7 TIME-SETTLEMENT CURVES FOR KANPUR SILT (Same final load intensity, loading period different)

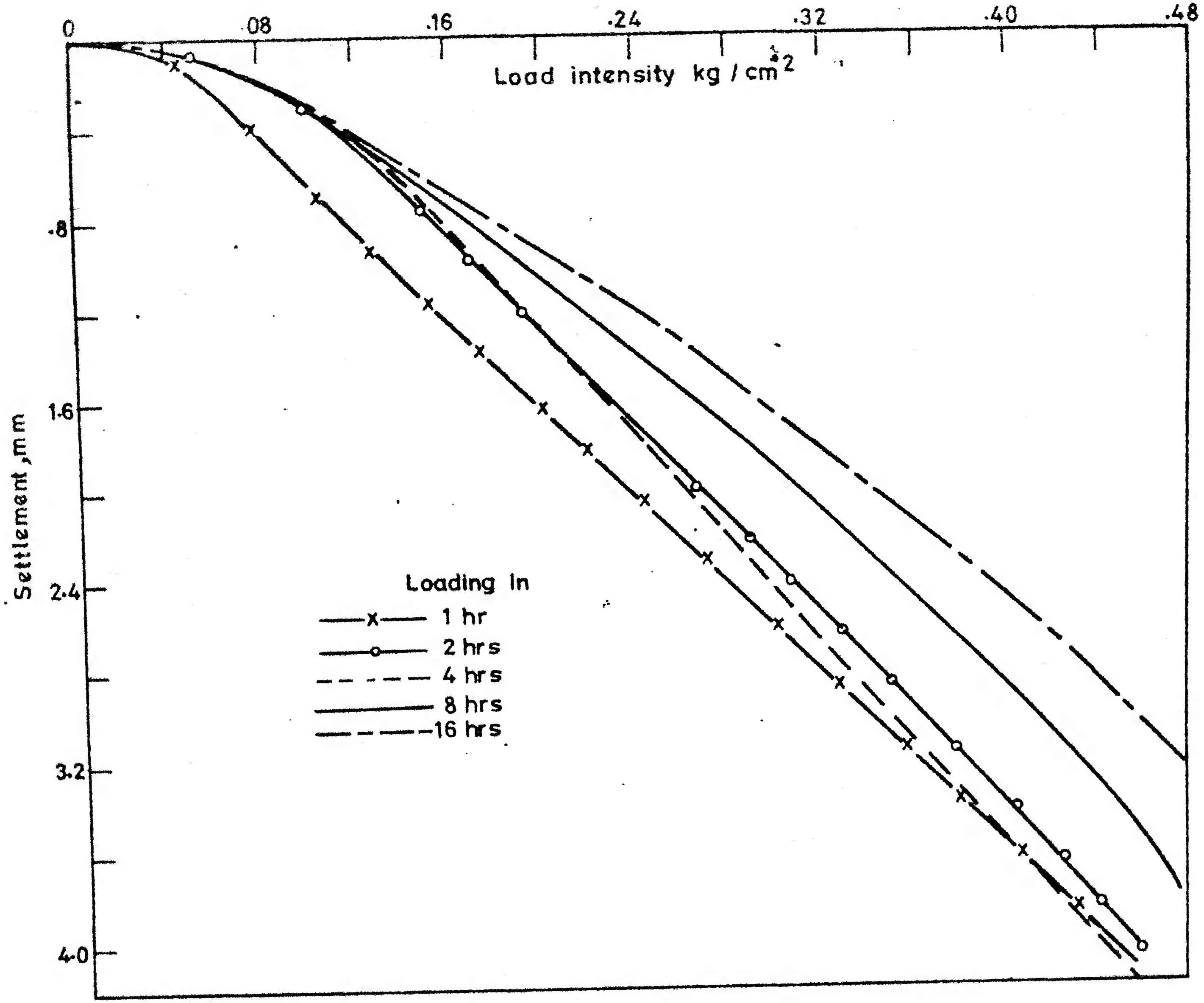


FIG. 8 LOAD INTENSITY ~ SETTLEMENT FOR KANPUR SILT (Same final load intensity, loading period different)

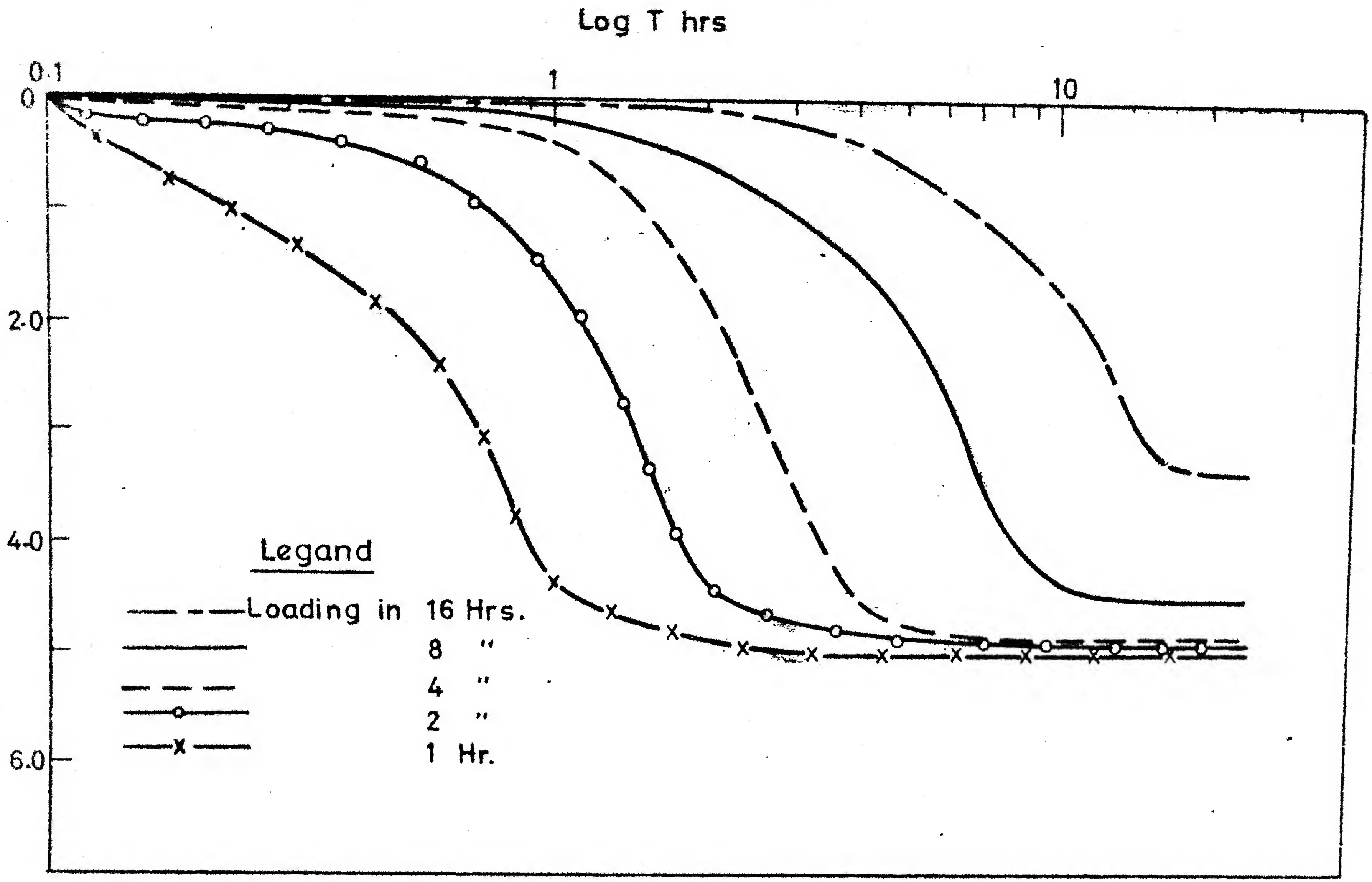
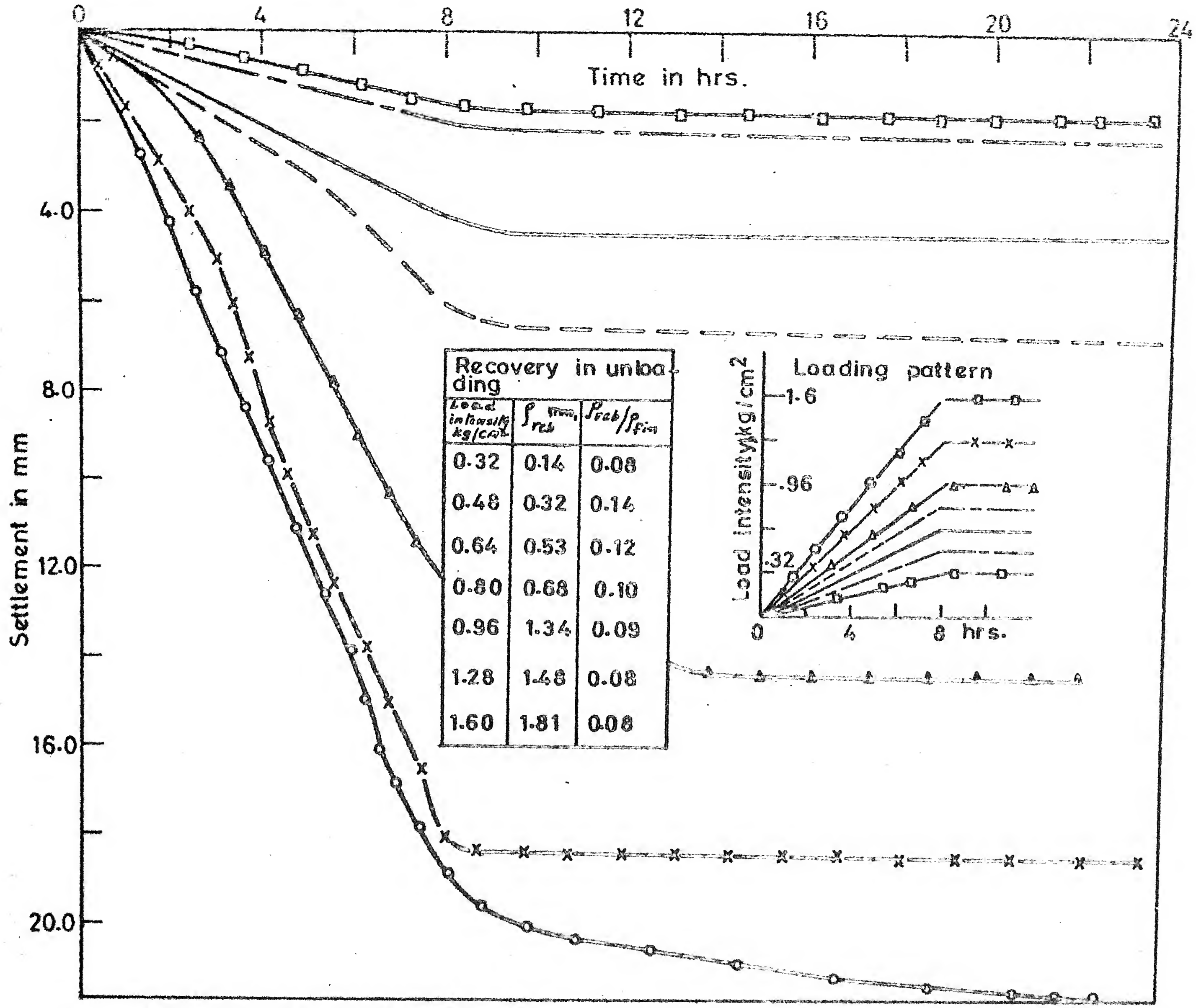


FIG. 9 SETTLEMENT – LOG.T FOR KANPUR SILT (Same final load intensity, loading period different)



Recovery in unloading		
Load intensity kg/cm^2	ρ_{reb} mm	ρ_{reb}/ρ_{fin}
0.32	0.14	0.08
0.48	0.32	0.14
0.64	0.53	0.12
0.80	0.68	0.10
0.96	1.34	0.09
1.28	1.48	0.08
1.60	1.81	0.08

FIG. 10 TIME SETTLEMENT CURVES FOR KANPUR SILT (Load intensity varying, loading time same)

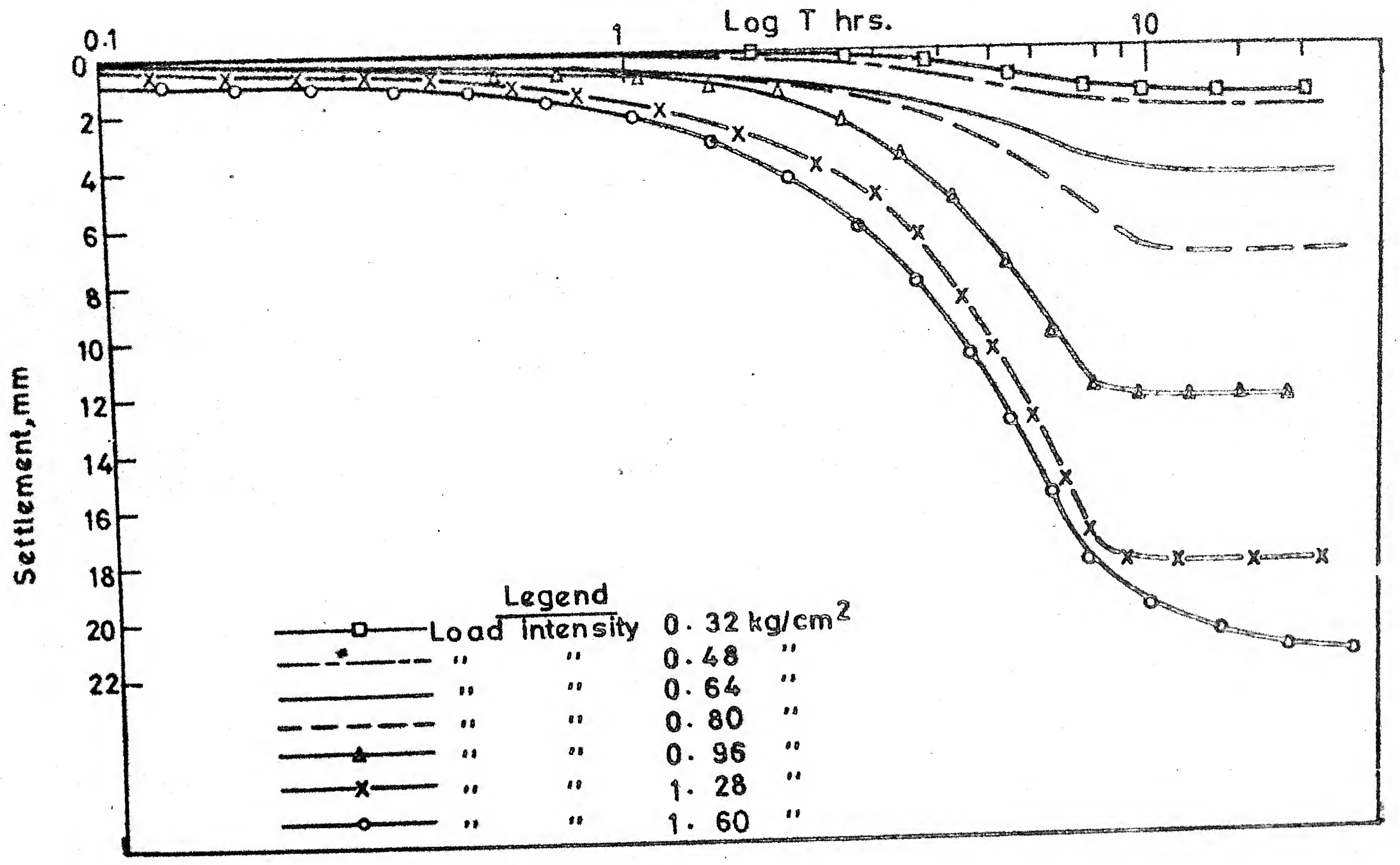


FIG. 11 LOG T-SETTLEMENT FOR KANPUR SILT (Load intensity varying, loading time same)

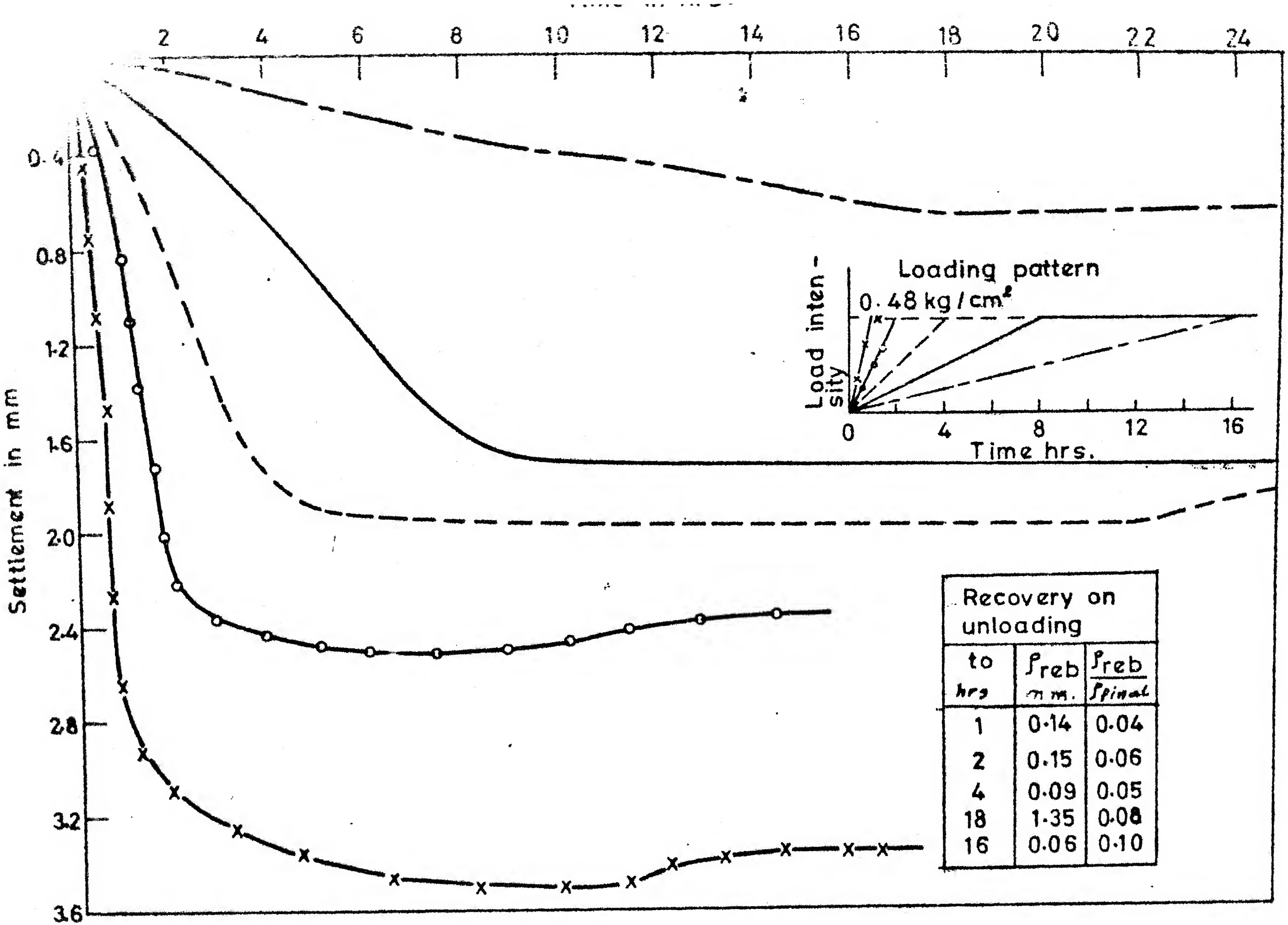


FIG. 12 TIME–SETTLEMENT CURVE FOR CLAY (Loading period varrying, same final load intensity)

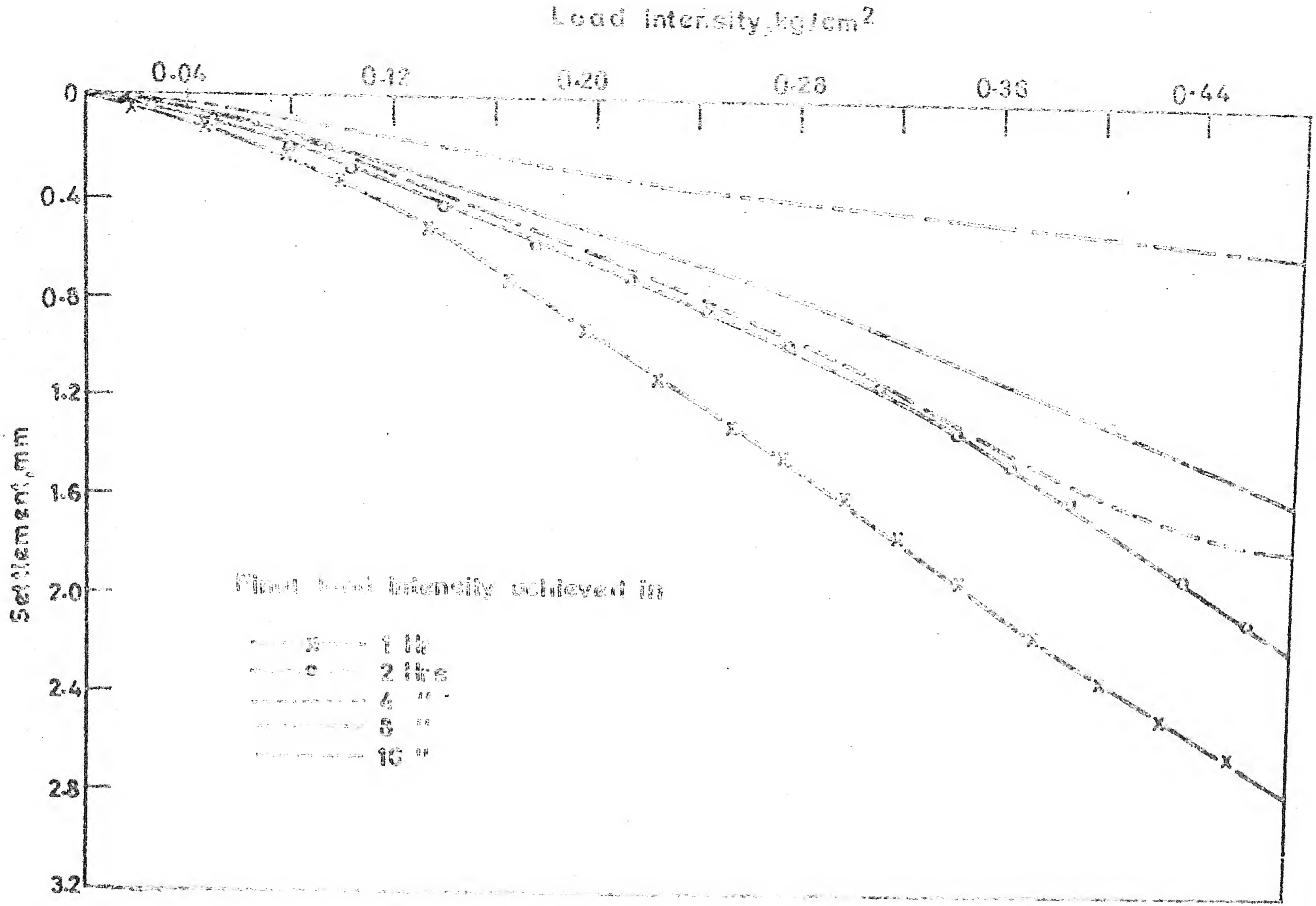


FIG. 13 LOAD INTENSITY - SETTLEMENT FOR CLAY (Loading period varying, same final load intensity)

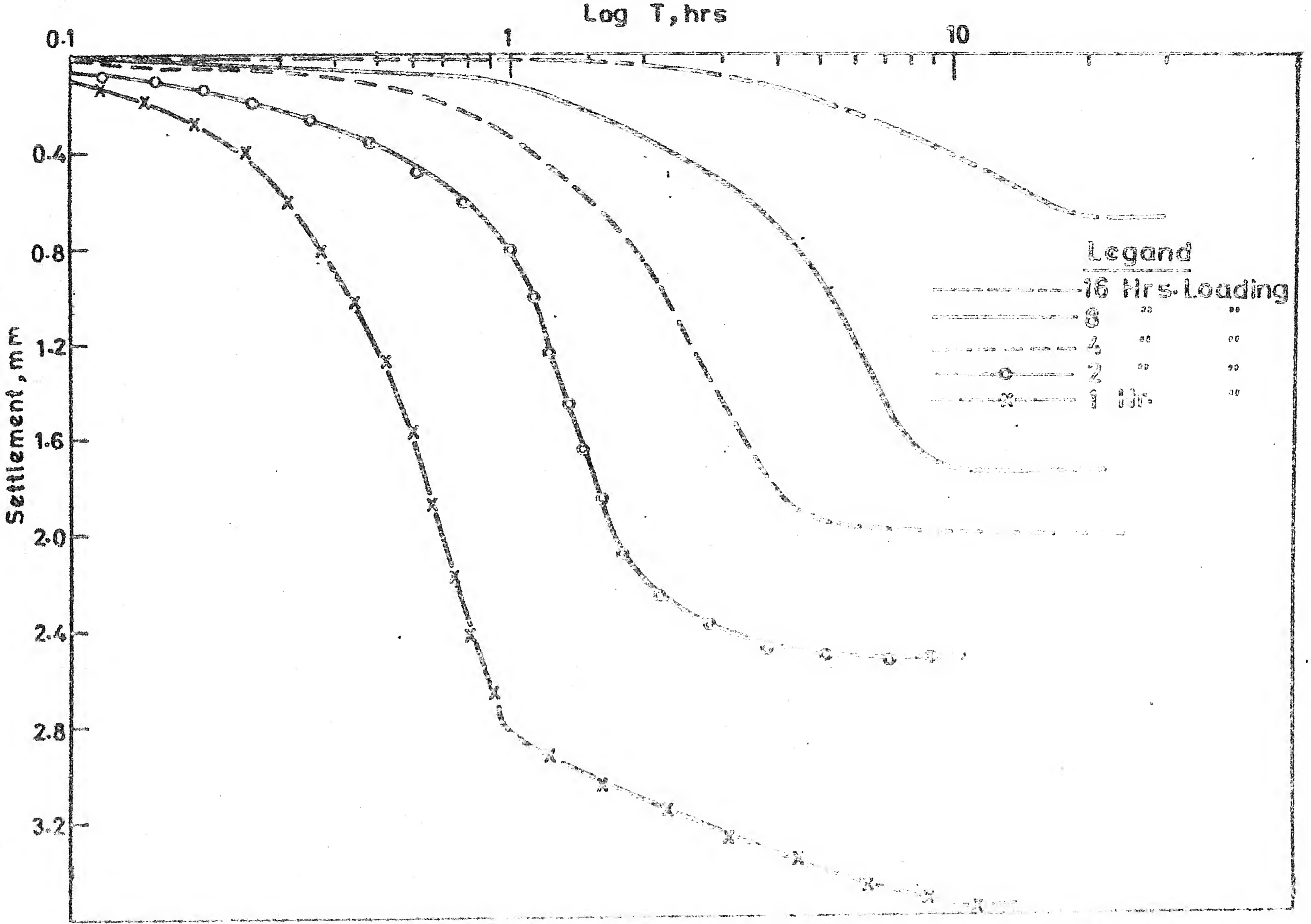


FIG. 14 LOGT - SETTLEMENT CURVES FOR CLAY (Loading period varying, same final load intensity)

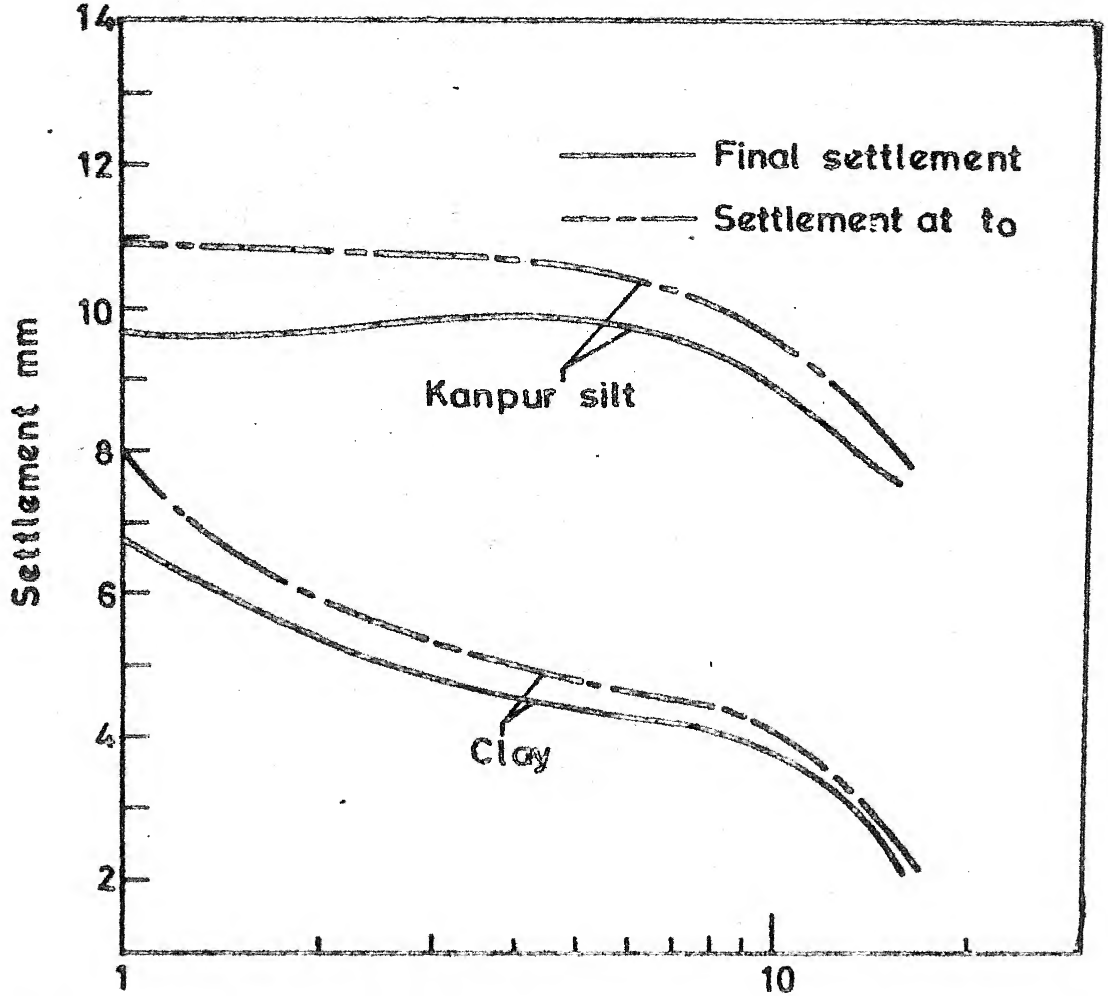


FIG. 15 LOG T – SETTLEMENT FOR KANPUR SILT AND CLAY
(Loading period different, same final load intensity)

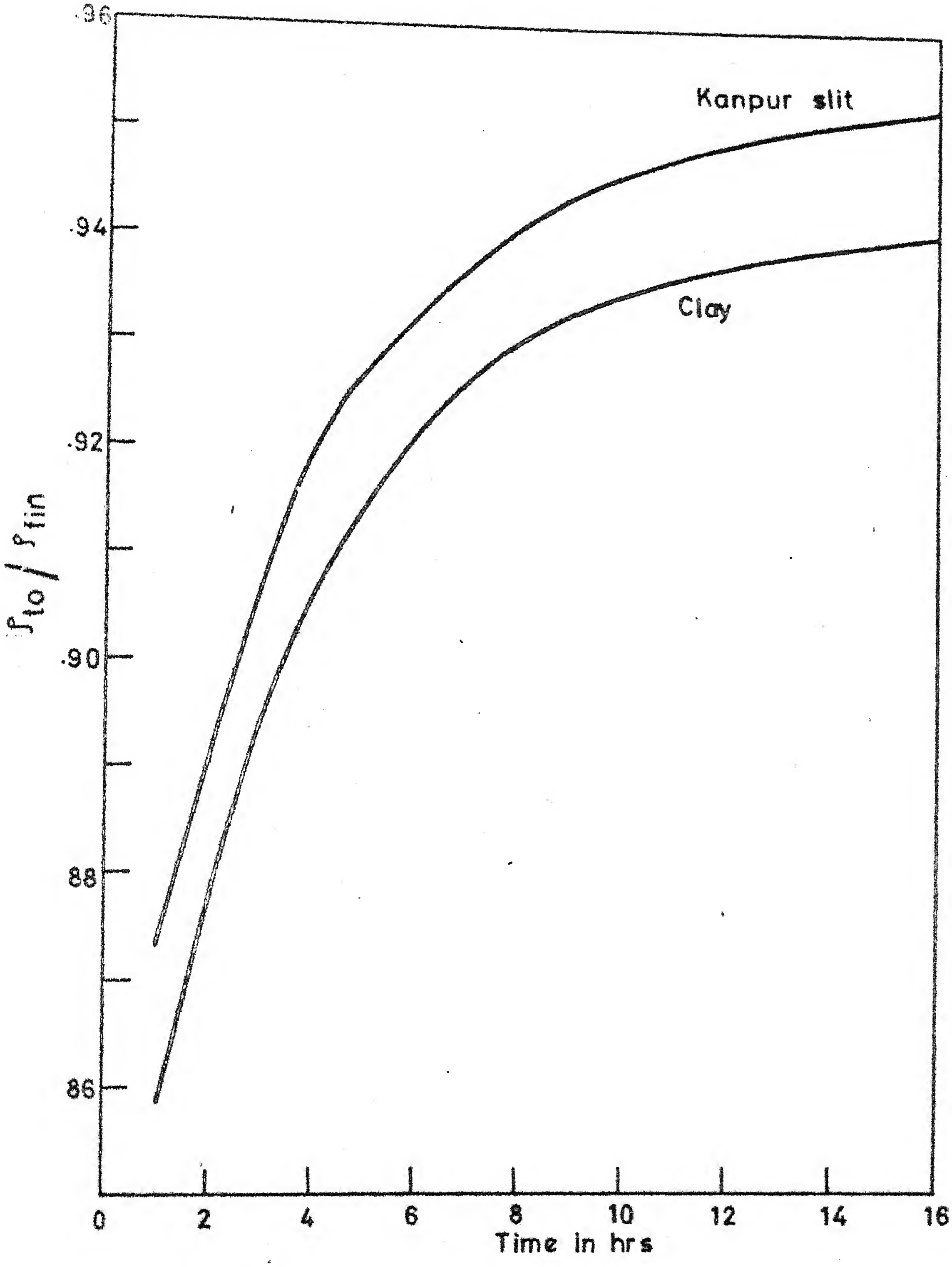


FIG. 16 ρ_{to} / ρ_{final} - to FOR KANPUR SILT AND CLAY
(Same final load intensity, loading period different)

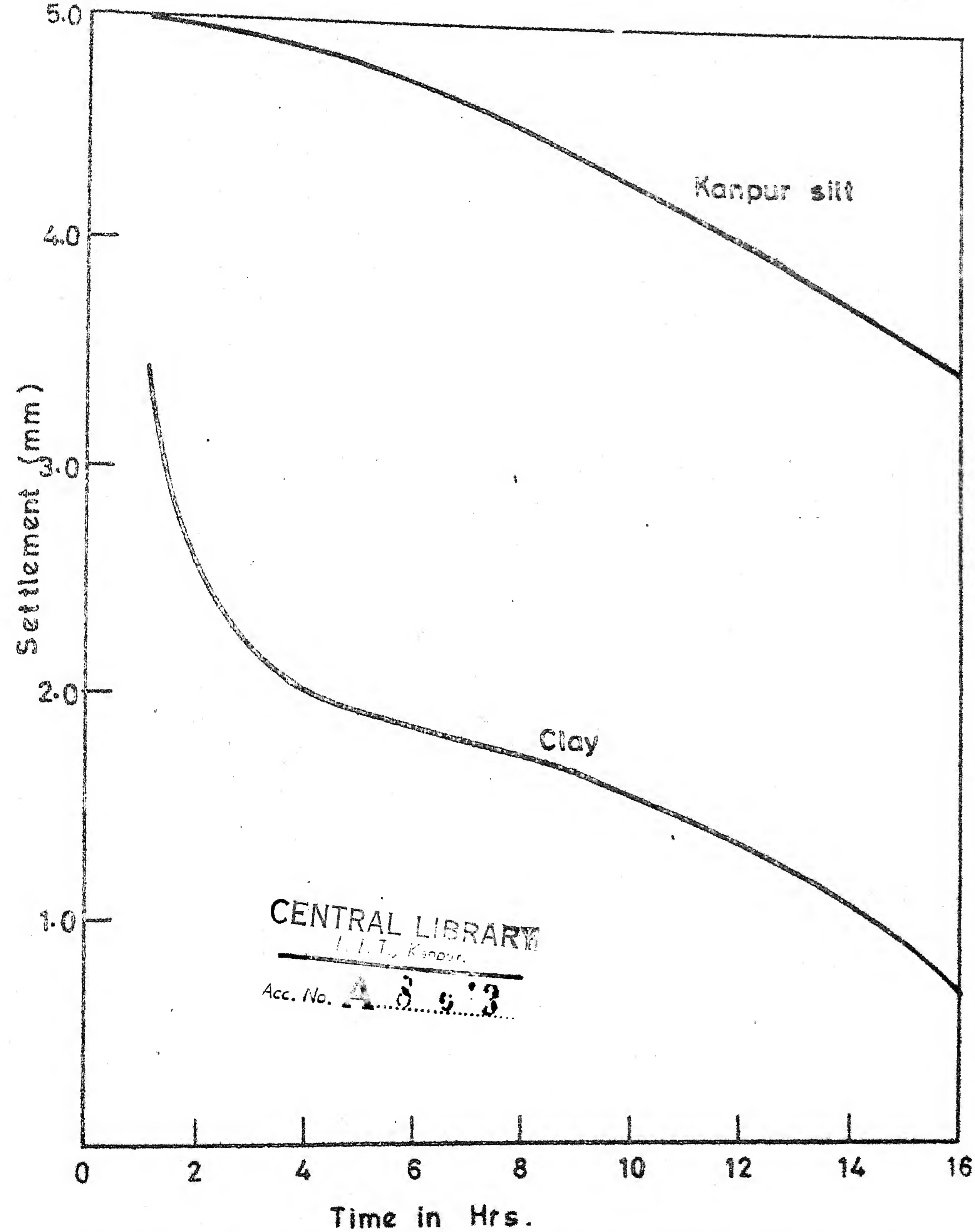


FIG. 17 FINAL SETTLEMENT-t_o FOR KANPUR SILT AND CLAY (Same final load intensity, loading period different)

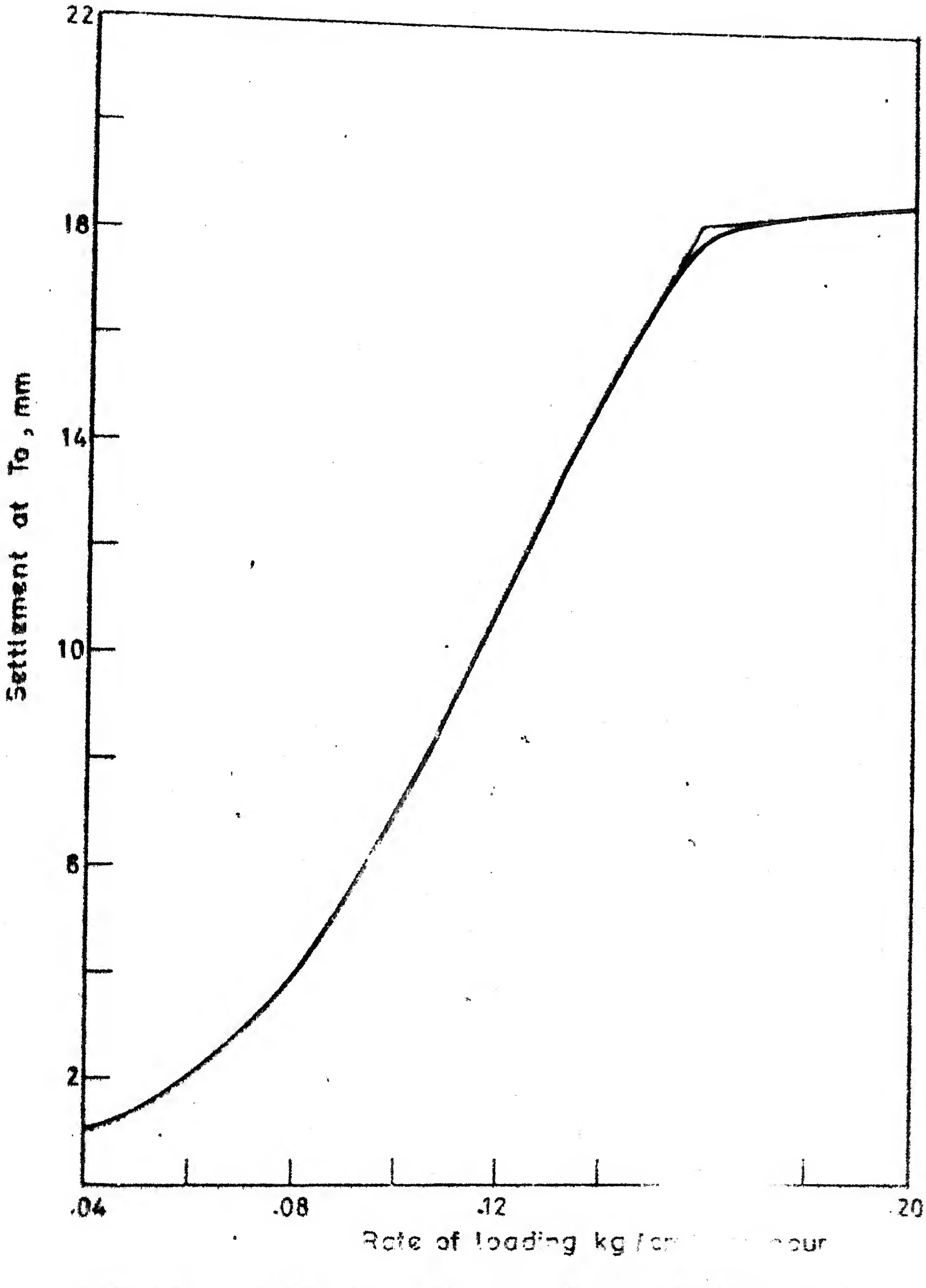


FIG. 18 RATE OF LOADING – SETTLEMENT AT Ta FOR KANPUR SILT (Load intensity varying, loading period same)

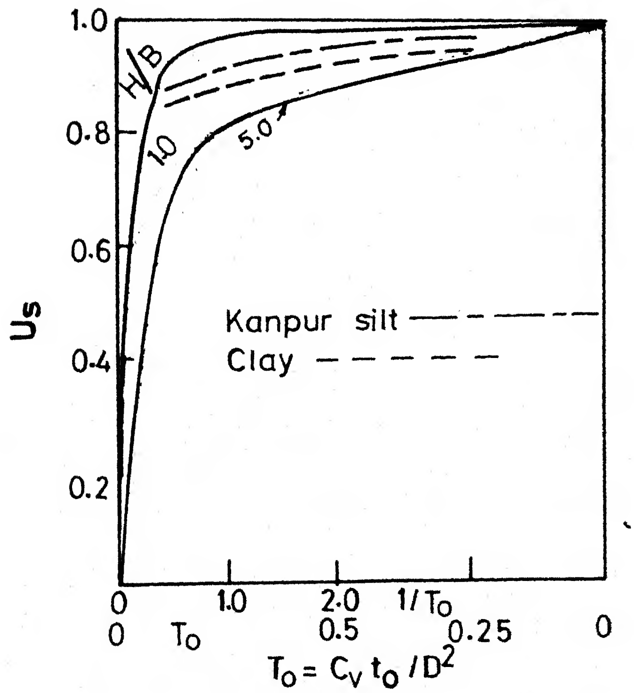


FIG.19 COMPARISON BETWEEN DESIGN CURVES AND EXPERIMENTAL CURVES [Madhav and Vitkar (1981)]

APPENDIX

PARAMETRIC STUDY OF BEHAVIOUR OF RAMMED STONE COLUMNS FROM FIELD DATA(LOAD TEST DATA)

1. Introduction:

On many occasions soil engineers are confronted with the problem of ground improvement in weak or loose soils. The ground improvement methods such as:

(i) Use of either friction or load bearing piles,

(ii) preloading along with sand drains or only preloading to pre-empty settlement and to stiffen the soil,

(iii) sand drains to accelerate the settlement,

(iv) replacing the soft or weak soil by stonger material,

(v) stone columns or granular piles.

They can be used independently or in some combination.

The first four methods are well accepted in practice. The use of stone columns or granular piles to support heavy loads was known to French military engineers as early as in the 19th century. Hughes and Withers (1974), report that as a by product of vibrofloatation technique for granular soils the use of stone column was rediscovered. Once again the stone columns are finding wider applications for supporting embankments, oil and water storage tanks,

founded on weak soils.

Stone columns have extensively been used in reinforcing clay deposits. Their use in noncohesive soil is not so common as recourse has always been taken to techniques like vibrofloatation, compaction piling, use of explosive charges etc. Stone columns are self adjusting to the soil condition to the extent that enlargement of the column during ramming or vibration depends on soil consistancy.

The stone column is essentially a system of soil reinforcement with the additional advantage of providing a drainage path. The stone column has the ability to adjust itself to the applied loads and to redistribute the load where stress concentration occurs. This is because there is no collapse as such,but only increase in deformation associated with bulging when the critical stress is exceeded.

Based on elastic theory, parametric study of the stone column load test data is made. Stone columns were of the rammed type. Field data of Raoli Hill area and Kandla fertilizer area,based on load test conducted by M/s Dubon Project Engineering,is considered for the study,

2. Technique and Installation Procedure:

In vibrofloat technique, soft cohesive strata is replaced by granular material which is packed by ramming or

vibration. A vibratory poker is advanced by jetting and the granular backfill is added through the annulus formed around the poker by water jetting. The granular fill is compacted by vibration so that a compacted sand and gravel column is left behind as vibratory poker is withdrawn.

In the rammed stone column technique,the granular fill is introduced into bored hole and compacted by heavy rammer through the hole. The cased bore hole may be advanced by any conventional boring method. The degree of compaction that can be achieved depends on several factors including the size of the hole; size, gradation and shape of granular fill, depth of filling, the weight and height of fall of the hammer, number of blows etc. Generally a gap graded mixture comprising of crushed stone of 20-25 mm size and medium sand (below 2 mm) is used. This avoids segregation and ensures better intergranular contact as the sand eventaaly fills into the void of the stone in a gap graded system.

After cleaning the bore hole a small quantity of granular material is placed in the casing and compacted by hammer till specified degree of compaction is achieved. A small quantity of granular fill is again added in the casing, and the tube is withdrawn for a short length at the same time, ensuring that the bottom end of the casing is always below the fill material. Granular material is then

compacted to the specified degree of compaction. The process of filling compacting and withdrawl of the tube is continued till the entire column is built up.

It is necessary to observe certain control measures in order to ensure uniform quality of columns with a dense state of packing and freedome from contamination. The degree of compaction is measured by a 'set' criterion. Good results can be obtained when ramming is done with water above the fill material although the compactive effort increases. Wet ramming has additional advantage of ensuring better integranular contact as aggregates get cleaned while descending through water and any intervening clay will get converted into a thin slurry. A slight increase in consumption can also be observed. Aggregate to sand is added in the ratio of 5:1 by volume.

3. Field Applications :Review of Case Studies:

Performance of structures supported by stone columns in weak soils are reported by field engineers. These case studies are reviewed here.

Hughes et al. (1975) have compared the field load bearing capacity of a stone column in soft clay with the prediction of its behaviour from laboratory model tests. The prediction is excellent and it is concluded that the stone column improves substantially the bearing capacity

of the natural soil.

Rathgeb and Kutzner (1975) have presented the performance of an embankment for a motorway on soft clay treated with stone columns.

For a sewage treatment plant on soft fine grained soil in an area of seismic susceptibility, improvement in strength with stone column was considered as a foundation solution by Engelhardt and Golding (1975). A series of large scale field tests on stone columns installed with vibrofloation equipment was conducted.

An interesting study on economical evaluation of embankment design has been made by Matsuo and Kuroda (1975). Four different alternatives are investigated to evaluate the method of design on cost function basis. It is shown that ,the alternative to increase shear strength by improving the poor ground with stone columns is profitable, for the case of high land cost. Datye (1981) has given comparitive cost for stone columns installed by various methods. He states that pressassembled stone columns are cheapest amongst stone column installed by vibrofloat, rammed technique and pre-assembled technique.

Datye and Nagraju (1975) present the behaviour of a foundation with stone columns at one of the fertilizer plants in India, where a large depth of soft compressible

deposit exists. Good agreement is recorded between the observed behaviour and the estimated capacity and settlements. Rao and Bhandari (1979) have observed from actual field study that the loose to medium dense cohesive statum exibited 60-280 percent heigher ultimate load carrying capacity with stone column. Madhav (1981) concluded from laboratory tests that with stone columns , the load carrying capacity of the soil increases by 100 - 200 percent.

From a case study Geenwood (1970) observed that even for a relative wide spacing of 2.3 m, the stone columns reduced settlements to 50 percent.Hughes et al. (1976) studied a case in which stone columns installed by vibrofloat. The increase in bearing capacity was of the order of 2.5 to 4 times. At shallow depths very substantial increase of bearing capacity due to stone column well compacted into clays, is reported.

A case where stone columns have had no effect on rate of settlement or amount of settlement is reported by MeKenna et al. (1976). The reason put forward is that only single size particles were used. Hence slurry got trapped in the voids and drainage channel were blocked. Also the installation technique might have remoulded the

adjacent soil damaging natural drainage path.

4. Field Data:

Field data available for Raoli Hill area consists of:

(1) Soil profile

(2) Diament of stone column

(3) Length of stone column

(4) Consumption of stone aggregates and sand

(5) Load test arrangement

(6) Load-settlement-time data.

Field data for Raoli Hill area is presented in Table No. A.I to Table No. A.IV and Figure No. A.I and Figure No. A.II.

For Kandla fertilizer area, available field data:

(1) Soil profile

(2) Diameter of stone column

(3) Length of stone column

(4) Load test arrangement

(5) Load-settlement data for single column and seven column load test.

Field data for Kandla fertilizer area is presented in Table No. A.V and Figure No. A.II and Figure No. A.III.

5. Method of Analysis:

1. From consumption of stone aggregates and sand the actual diameter of the stone column per lift is worked out.

2. Ultimate capacity of the stone column is worked out from load-settlement data.
3. For K varing between 500-1000, Eusoil is worked out using elastic solution for piles.
4. Considering elastic solutions for group settlement of piles is applicable to stone columns, group settlement of stone columns is estimated and compared with actual.
5. Ultimate capacity of stone column using pile formula and cavity expansion theory of Vesic (1972) is worked out and compared with the designed capacity.

6. Specimen Calculations:

From laboratory tests it was observed that one m^3 of loose sand and aggregate mix when compacted, occupies 0.8 m^3 or there is 20 percent reduction in volume, due to compaction.

6.1 Roali Hill Area:

<u>Stone Column No. 11:</u>

1) Actual diameter of stone column from consumption for stage 3:

From 8.0 to 4.3 mts. a length of 3.7m.,52 cft. of aggregate and 10.4 cft. of sand was added. Total 62.4 cft. when compacted 49.92 cft. Considering 62.5 cm dia and 3.7 m height. Theoretical volume should have been 40.09 cft. Hence actual diameter:

$$\frac{\pi}{4} d^2 \times 3.7 = 49.92 \times 0.028317$$

$$d = 0.7 \text{ mts.}$$

The average diameter for the entire column on similar lines worksout to 75 cm.

2) By drawing a tangent to load settlement curve as shown in figure the capacity of stone column curve was estimated to 21.6 T. It is designed for 24 T.

3) Even though the tip of the stone column rests on firm ground, due to the flexibility of the stone column the load transfer is not due to end bearing, also E_b is unknown. Hence stone column is considered as a floating pile.

$$\rho = \frac{PI}{E_s d}$$

$I = I_o \, R_k \, R_h \, R.$

ρ = settlement of pile head.

P = applied axial load.

I_o = settlement influence factor for incompressible pile in semiinfinite mass for $\nu = 0.5$

R_k = correction factor for pile compressibility.

R_h = correction factor for finite depth of a layer on a rigid depth.

R = correction factor for soil Poisson's ratio.

h = total depth of soil layer.

Length of column is 12.3 but for load test it is 12.3- 1.2 m = 11.1 m (see load test arrangement) Fig. No. A-V).

Two diameters are considered 62.5 cm and 75 cm.

$\nu = 0.5$, $R_{\nu} = 1.0$

For K = 500.

Dia cm.	I_o	R_k	R_h	I	I/d	L/d
62.5	0.095	1.15	0.59	0.0322	0.05757	17.76
75.0	0.100	1.10	0.58	0.0319	0.0425	14.80

For 4T load;

$$\rho = 0.02 \text{ inch} = 0.0508 \text{ cm} = 5.08 \times 10^{-4} \text{ m}$$

$$5.08 \times 10^{-4} = \frac{4 \times 0.0322}{E_s \times 0.625}$$

$$E_s = 407.64 \text{ T/m}^3$$

Similar calculations are carried out for all load values .

In load settlement data, values of settlement after 'stabilization' are given, hence values of E_s drained are also obtained.

E_s in t/m^2 for stone column No. 11 are tabulated in Table No. A.II.

4) i) Stone column capacity by treating stone column as a

pile driven in clays and clayey silt.

$Q_b = C_b\ N_c\ A_b$; $Q_s = \alpha\ \bar{C}_d$

$= 2.4 x 9 x \frac{\pi}{4}\ (0.625)^2$ $Q_s = 0.5 x 1.66\ x\ \pi(0.625) 12.3$

$= 6.63$ $= 1.63\ x\ 12.3$

$Q_{ultimate} = 26.68$ T $= 20.05$

Considering 75 cm dia.

$Q_b = 9.54$ $Q_s = 23.985$ $Q_{ult} = 33.525$ T

ii) Using cavity expansion factors

From consumption analysis for stone column no.11 it can be seen that between 2.5 to 2.3 maximum consumption is observed. Hence the pressure required to cause the expansion of cavity (as explained by Vesic (1972)) at 2.4 m depth is considered.

$$P_u = C\ F'_c + q\ F'_q$$

For the site $\gamma_{sub} = 0.8$ T/m^2 $C = 1.11$ t/m^2 $\phi = 0$

and $K_o = 0.6$ (assumed).

Mean normal stress at 2.4 m below ground

$$= \frac{1}{3}\ (2.4\ x\ 0.8 + 2\ x\ 0.6\ x\ 2.4\ x\ 0.8)$$

$$= 1.408\ t/m^2$$

P_u estimated for two extreme values of I_r

$I_r = 10$; $\phi = 0$; $F'_c = 3.3$ and $F'_q = 1.0$.

$P_u = 3.3 \times 1.1 + 1.0 \times 1.408 = 5.038\ t/m^2$

$I_r = 300 \quad \phi = 0 \quad F'_c = 6.7$ and $F'_q = 1.0$

$P_u = 1.1 \times 6.7 + 1.408 \times 1 = 8.778\ t/m^2.$

Ultimate vertical stress is taken as 6 P_u.

$$\text{Ultimate load} = 5.038 \text{x} 6 \times \frac{\pi}{4} (0.625)^2$$
$$= 9.274 \text{ T}$$
$$= 13.35 \quad (\text{dia } 75 \text{ cms})$$

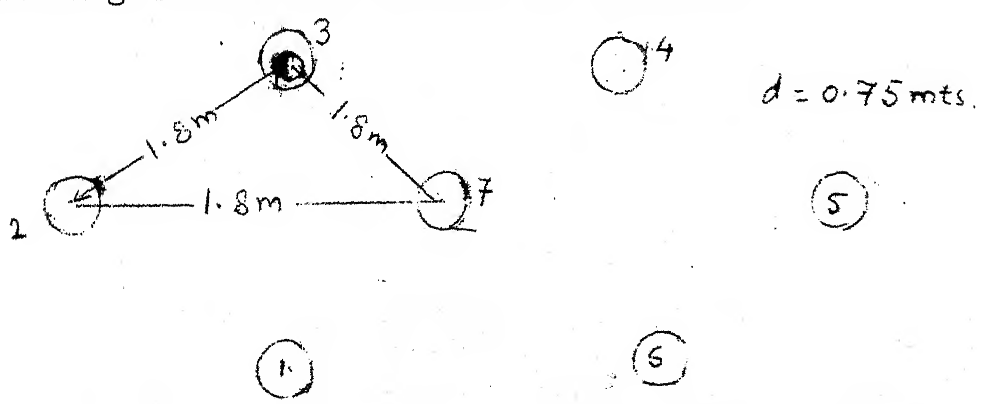
$$\text{For } I_r{=}300 \quad P_u = 8.778 + 6 \times \frac{\pi}{4} (0.625)^2$$
$$= 16.16 \text{ T}$$
$$= 23.27\text{t} \ (\text{dia } 75 \text{ cm})$$

6.2 Kandla Fertilizer Area:

Based on the analysis for E_u on lines described earlier, the stone column capacity is calculated. The calculations for group settlement are given below:

$K = 500 \qquad L/D = 11.07.$

From Figs. 6.2 and 6.3 Poulos and Davis (1980) following interaction factors are obtained.



Stone columns 1-6 behave in similar fashion (Group A) stone column 7 behaves in another fashion (Group B).

	Stone column 1 (type A)		Stone column 7 (type B)	
j	s/d	α_{ij}	s/d	α_{ij}
1	0	-	2.4	0.49
2	2.40	0.49	2.4	0.49
3	4.16	0.36	2.4	0.49
4	4.80	0.32	2.4	0.49
5	4.16	0.36	2.4	0.49
6	2.40	0.49	2.4	0.49
7	2.40	0.49	-	-

Settlement of stone column 1 is given by :

$$\rho_A = \rho_1 \left[P_A\ (0.49+0.36+0.32+0.36+0.49) + P_B(0.49) + P_A\right]$$

$$\frac{\rho_A}{\rho_1} = 3.02\ P_A + 0.49\ P_B$$

where ρ_1 is settlement of single stone column under unit load, and P_A and P_B are loads on stone column belonging types A and B respectively:

Similarly for stone column 7 ,

$$\rho_B = \rho_1 \left[P_A(0.49)\ 6 + P_B\right]$$

$$\frac{\rho_B}{\rho_1} = 2.94\ P_A + P_B \qquad (2)$$

For rigid cap $\rho_A = \rho_B$ $P_A = 6.375\ P_B$ from (1) and (2).
From Table A.V average $\rho_1 = 2.5\text{x}10^{-4}$ mts/tonne.

For any system of load P it will be shared in the following manner $6P_A + P_B = P$ $39.25\ P_B = P$

If $P = 100$ $6P_A+P_B = 100$ and $P_A = 6.375\ P_B$

$P_B = 2.550$ T $P_B = 16.23$ T

Substituting $P_A = 6.375$ in (1)

$$\frac{\rho_A}{\rho_1} = 3.02(7\ P_B) + 0.49\ P_B = 21.63\ P_B$$

$$\rho_A = 19.7725(2.55)\ (2.5\text{x}10^{-4})$$

$$= 0.0179 \text{ mts} = 17.9 \text{ mm}$$

Similarly settlement is **computed** for K = 1000.

7. Results:

7.1 Stone column diameter based on consumption of material:

For the two stone columns that were considered in Raoli Hill area , the average diameter for the entire column No. 11 works out to 75 cm and that of stone column No. 12 works out 85.5 cm. Actual diameter of casing installed 62.5 cm.

For stone column 11, the diameter based on consumption analysis varied from 43 cm to 81 cm. Less

diameter indicating either the hole at 12.3 to 10.8 is not cleaned properly or there is blow at that level. Maximum diameter 1.81 mts observed at 2.5 - 2.3 m level, indicating weak layer.

For stone column 12, the diameter based on consumption analysis varied from 52 cm to 145 cm. Diameter less than the bored diameter, indicates, that there is blow at 9.0 - 7.4 m level or the hole is not cleaned properly. Maximum diameter of 1.45 m observed at 3.6-4.2 m level.

7.2 Load carrying capacity based on load-settlement data:

1) For stone column 11 , Raoli Hill area ,the estimated capacity is 21.6 Tonnes.

2) For stone column 12, Raoli Hill area the estimated capacity is 18 Tonnes.

3) For Kandla fertilizer area, the estimated capacity is 16 Tonnes.

Stone columns in Raoli Hill area were designed for 24 Tonnes and in Kandla fertilizer area 30 Tonnes.

7.3 E_{soil}:

For Raoli Hill area, for Kranging from 500-1000 , computed E_{soil} based on immediate settlement (E_s undrained) works out in the range of 400 to 32 T/m^2. Computed E_{soil} based on stabilised value of settlement (E_s drained) works out in the range of 300 to 25 T/m^2.

For Kandla fertilizer area, computed E_{soil} based on settlement works out in the range of 975 to 155 t/m^2 for K ranging from 5000 to 1000.

7.4 Group settlement:

For K varying from 100-1000, the computed group settlement and observed settlement are plotted in Fig. A.III.

7.5 Stone column load capacity:

Area	Pile Formula Tonnes		Cavity expansion theory				
			I_r=10		I_r=300		Failure depth (assumed)
Raoli Hill stone column 11	d=62.5 26.68	d=75cm 33.53	d=62.5 9.27	d=75cm 13.35	d=62.5 16.16	d=75cm 23.27	2.4 mts below G.W.
Raoli Hill stone column 12	d=62.5 21.3	d=85.5 cm 32.195	d=62.5 cm 11.00	d=85.5 cm 20.59	d=62.5 cm 17.89	d=85.5 cm 33.47	4.0 mts below G.W.
Kandla Fertilizer Area	d=75 cm 62.9		d =75cm 24.49		d=75 cm 42.52		4.0 mts below G.W.

8. Conclusion:

Due to the ramming the average increase in stone column diameter is observed. It is possible to use load-settlement data to estimate stone column load carrying capacity. Estimation is good in Raoli Hill area but not so in Kandla fertilizer area. Range of K c nsidered and E_s computed is quite large, but due to limited data, smaller range cannot be considered. The same is true for computation of settlement of group. Pile formula over-estimates the capacity in Kandla area. The prediction is good in case of Raoli Hill area , cavity expansion theory , using K = 300 (stiff clay),estimates the capacity in the viscinity of the design capacity only if the enlarged diameter based on consumption is used. If I_r = 10 (soft clay) is used the estimated capacity falls short of the designed. For capacity calculation, if the diameter of casing is used for both I_r = 10 and 300, the computed capacity is less than the designed. For Kandla Fertilizer area, I_r=10 estimates the capacity, which is near to the designed.

TABLE A.I : RAOLI HILL AREA STONE COLUMN NO. 11

Length of stone column 12.3 mts. Diameter of casing (O.D.) 62.5 cm.

G.W.L. 1.0 m below G.W. Weight of rammer 2.0 T. Height of fall 1.0 m

Set 20 blows for 1.0 cm penetration.

CONSUMPTION RECORD

Stage No.	From - To	Depth	Consumption in cft		Total cft	0.8 of total m^3	Theoretical vol. m^3	Diameters based on consumption mts.
			Sand	Aggregates				
1	12.3 - 10.8	1.5	8	1.6	9.6	0.2175	0.4602	0.430
2	10.8 - 8.0	2.8	40	8.0	48.0	1.1000	0.8590	0.710
3	8.0 - 4.3	3.7	52	10.4	62.4	1.4100	1.1350	0.696
4	4.3 - 2.5	1.8	29	5.8	34.8	0.7880	0.5520	0.746
5	2.5 - 2.3	0.2	19	3.2	22.8	0.5165	0.0610	1.810
6	2.3 - 1.4	0.9	11	2.2	13.2	0.2990	0.2761	0.650
7	1.4 - 1.2	0.2	12	2.4	14.4	0.3262	0.0610	1.440
8	1.2 - 0.0	1.2	27	5.4	32.4	0.7340	0.3682	0.8820

TABLE A.II : ROALI HILL STONE COLUMN NO. 11

E_{SOIL} in T/m^2

Load Tonnes	Settlement mts.		K = 500				K = 1000			
	Immediate	Stabilised	d=0.625 m.		d =0.750 m.		d=0.625 m		d = 0.750 m.	
			$E_{undrained}$	$E_{drained}$	$E_{undrained}$	$E_{drained}$	$E_{undrained}$	$E_{drained}$	$E_{undrained}$	$E_{drained}$
4	5.08×10^{-4}	7.62×10^{-4}	406.063	270.71	334.65	223.10	370.72	247.15	313.62	209.08
8	1.11×10^{-3}	1.40×10^{-3}	371.680	294.69	306.31	242.86	339.33	269.04	287.06	227.60
12	2.70×10^{-3}	4.14×10^{-3}	221.010	149.50	182.14	124.39	201.78	137.80	170.70	116.58
16	5.60×10^{-3}	7.10×10^{-3}	147.34	116.21	121.43	95.77	134.52	106.10	113.77	89.74
20	9.40×10^{-3}	12.45×10^{-3}	106.33	82.84	90.43	68.27	100.17	75.63	84.74	63.98
24	1.5×10^{-2}	2.26×10^{-2}	82.51	54.76	68.00	45.13	75.33	50.00	63.73	42.30
28	2.5×10^{-2}	3.56×10^{-2}	57.76	40.56	47.60	33.43	52.73	37.03	44.61	31.33
32	3.94×10^{-2}	5.03×10^{-2}	41.88	32.81	35.52	27.04	38.24	29.95	32.35	25.34

TABLE A.III : RAOLI HILL AREA STONE COLUMN 12

Length of stone column 9.0 mts. Diameter of casing (O.D.) = 62.5 cm.
Weight of rammer 2.0 T, Height of fall 1.0 m
set 20 blows for 1.0 cm penetration

CONSUMPTION RECORD

Stage No.	From - To	Depth	Consumption cft		Total cft	0.8 total m^3	Theoretical m^3	Diameter based on consumption mts
			Sand	Aggregate				
1	9.0 - 7.4	1.6	2.0	10	12.0	0.3398	0.4910	0.5200
2	7.4 - 6.9	0.5	2.0	10	12.0	0.3398	0.1534	0.9302
3	6.9 - 4.2	2.7	5.4	27	32.4	0.9175	0.8283	0.6580
4	4.2 - 3.6	0.6	5.8	29	34.8	0.9854	0.18411	1.4460
5	3.6 - 2.6	1.0	2.2	11	13.2	0.3738	0.30681	0.6900
6	2.6 - 1.7	0.9	3.2	16	19.2	0.5440	0.2761	0.8850
7	1.7 - 0.0	1.7	9.8	49	58.8	1.6650	0.5215	1.1160

TABLE A.IV : RAOLI HILL AREA STONE COLUMN 12 CALCULATIONS FOR E_{SOIL} IN T/M

Load Tonnes	Settlements in mts.		K=500				K=1000			
			d=0.625 m		d=0.855 m		d=0.625 m		d=0.855 m	
	Imme-diate	Stabli-sed	$E_{undrai-ned}$	$E_{drai ned}$	$E_{undrai-ned}$	$E_{drai-ned}$	$E_{undrai-ned}$	$E_{drai-ned}$	$E_{undrai-ned}$	$E_{drained}$
4	1.657×10^{-3}	2.032×10^{-3}	174.682	141.93	167.17	135.83	166.30	135.51	156.07	126.81
8	3.556×10^{-3}	4.064×10^{-3}	162.200	141.93	155.23	135.83	154.42	135.51	144.70	126.81
12	6.35×10^{-3}	6.604×10^{-3}	136.252	131.01	130.39	125.38	129.71	124.72	121.74	117.06
16	8.89×10^{-3}	1.016×10^{-2}	129.760	104.30	124.18	108.66	123.54	108.10	115.94	101.45
20	1.245×10^{-3}	1.45×10^{-2}	115.82	99.45	110.83	95.17	110.26	94.67	103.48	88.85
24	1.702×10^{-2}	2.36×10^{-2}	101.67	73.32	97.30	70.17	96.90	69.80	90.84	65.57
28	2.794×10^{-2}	4.71×10^{-2}	72.25	42.95	69.25	41.91	68.78	40.89	64.56	38.38

TABLE A. V : KANDLA FERTILIZER STONE COL. NO. C-6

Load tonnes	Settlements mts	E_{Soil} (undrained)		Settlement load mts/tonne	P Tonnes	K = 500		K = 1000	
		K =500	K=1000			$P_B = \frac{P}{39.25}$	Settlement mm $\rho_A = 19.7425\, P_B \rho_1$ (group)	$P_B = \frac{P}{74.5}$	Settlement $\rho_A = 38.485\, P_B \rho_1$ (group)
4	3.5×10^{-4}	974.86	914.29	0.875×10^{-4}	50	1.274	6.29	0.671	6.45
8	1.0×10^{-3}	682.40	640.00	1.250×10^{-4}	100	2.550	11.95	1.340	12.89
12	2.3×10^{-3}	445.04	417.39	1.920×10^{-4}	150	3.820	18.86	2.013	19.37
16	4.2×10^{-3}	324.95	304.76	2.625×10^{-4}	200	5.696	25.15	2.685	25.83
20	7.0×10^{-3}	243.71	228.57	3.500×10^{-4}	250	6.370	31.44	3.356	32.29
24	1.04×10^{-2}	196.85	184.62	4.330×10^{-4}	300	7.643	37.73	4.027	38.74
28	1.45×10^{-2}	164.72	154.48	5.188×10^{-4}	350	8.14	44.01	4.700	45.20

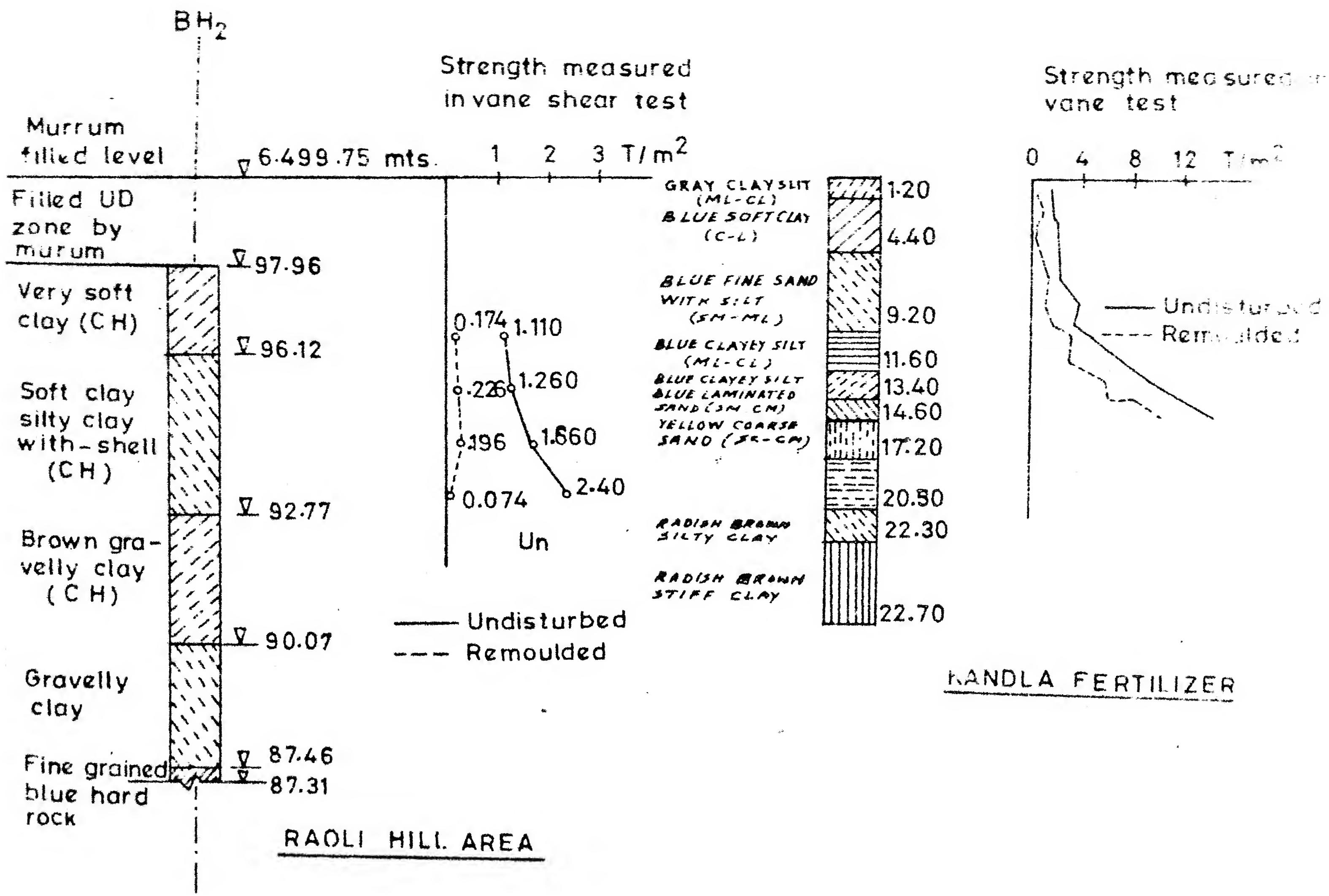


FIG. A-1 TYPICAL SOIL PROFILE

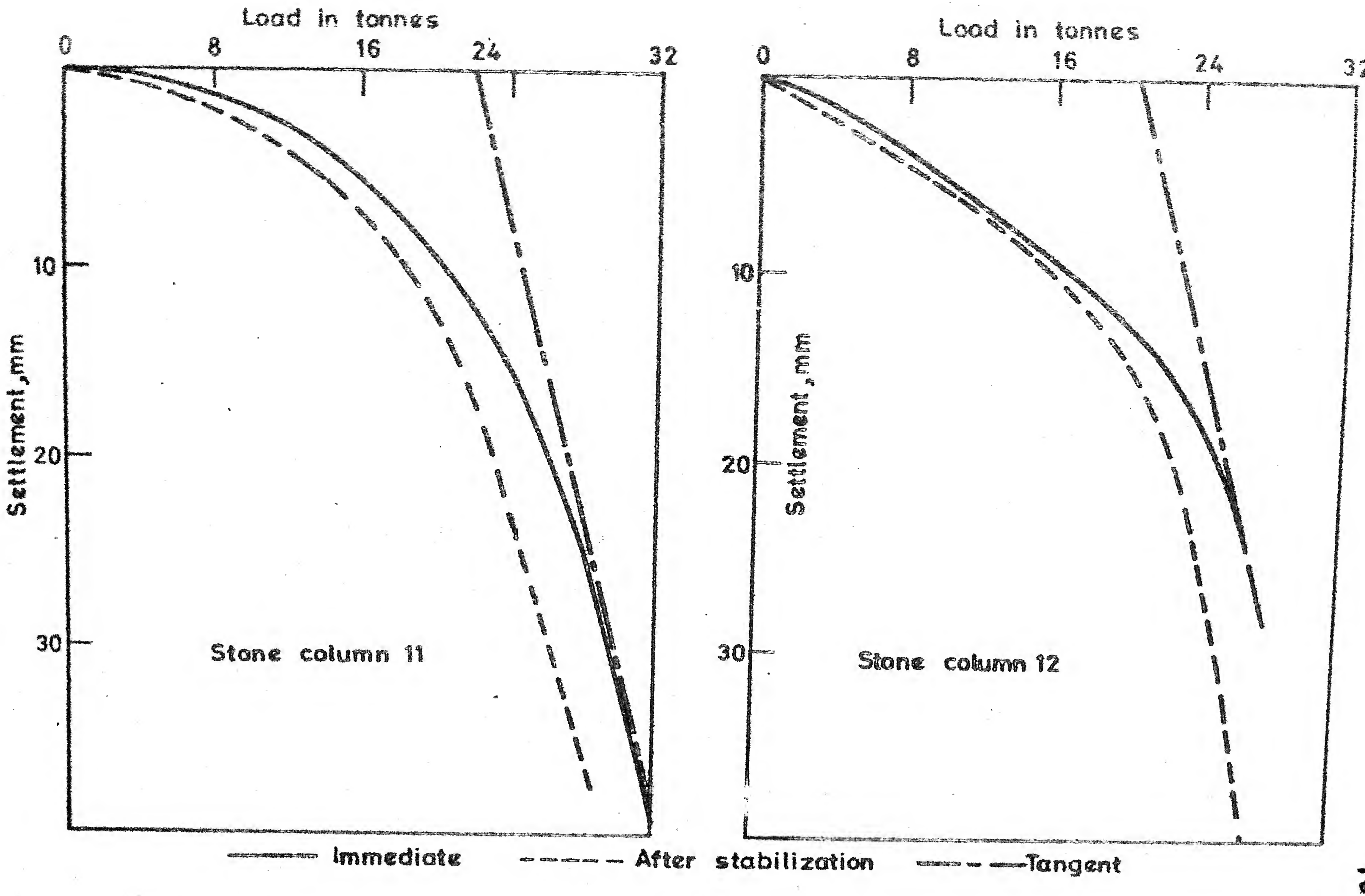


FIG. A-II LOAD SETTLEMENT CURVES FOR STONE COLUMNS IN RAOLI HILL AREA

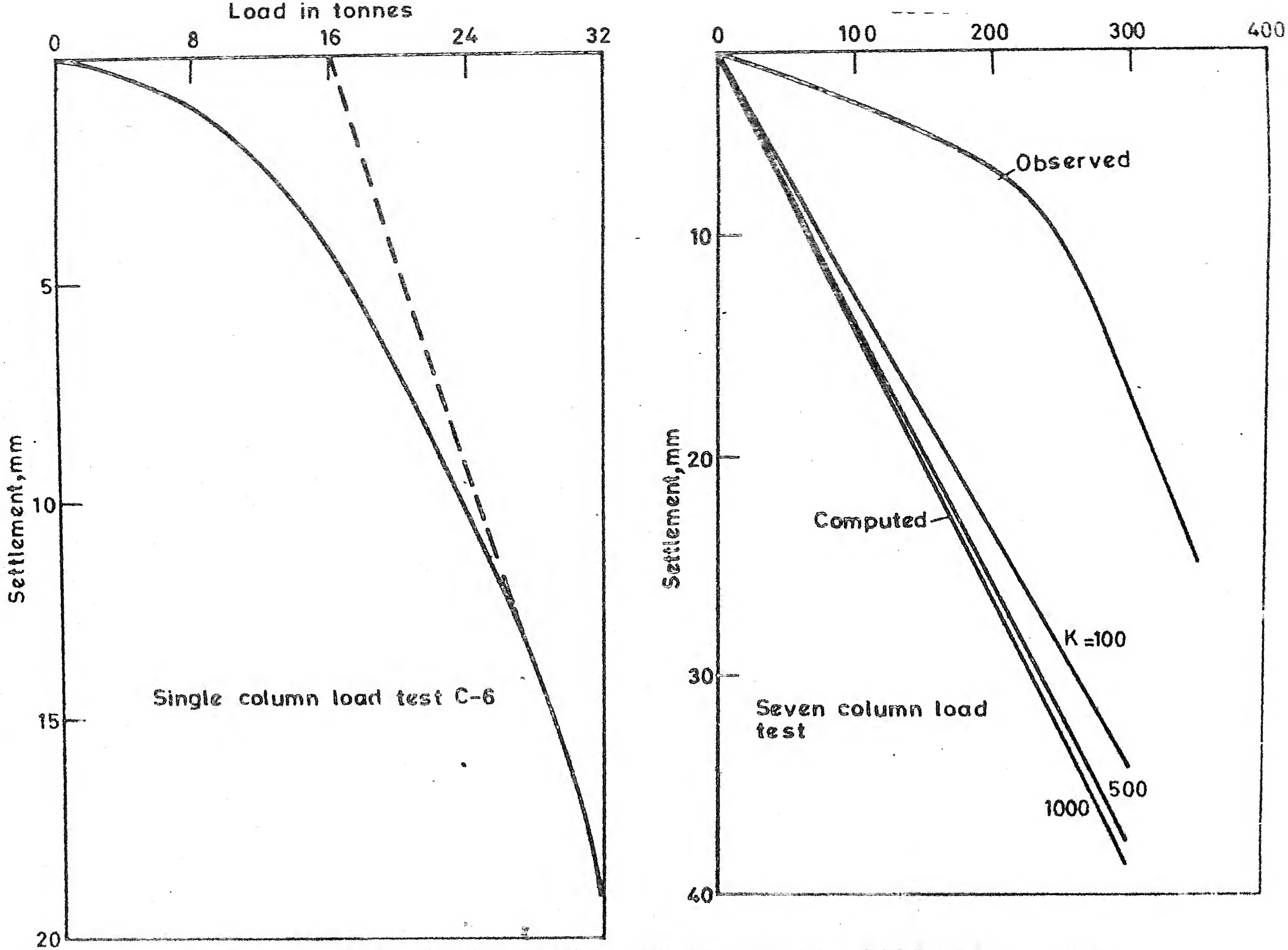
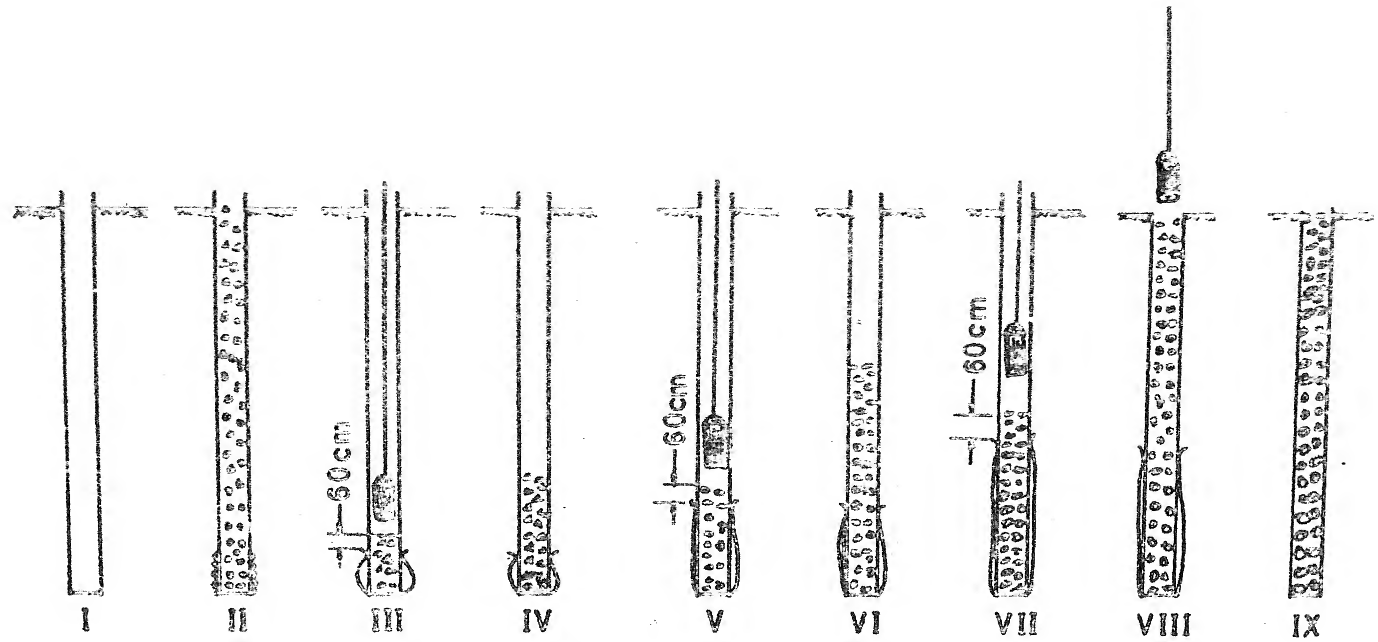


FIG. A-III LOAD SETTLEMENT DATA FOR KANDLA FERTILIZER AREA



I) Boring a cased hole upto desired depth

II) Filling aggreate and sand

III) Raming the matl. with sufficient overlap of casing

IV) Casing withdrawl and raming

VI) Filling aggregate and sand

VII) Casing withdrawl and raming

VIII) Filling aggregate upto top withdrawl of casing and raming with free fall

IX) Ready column

FIG. A-IV INSTALLATION OF RAMMED STONE COLUMNS

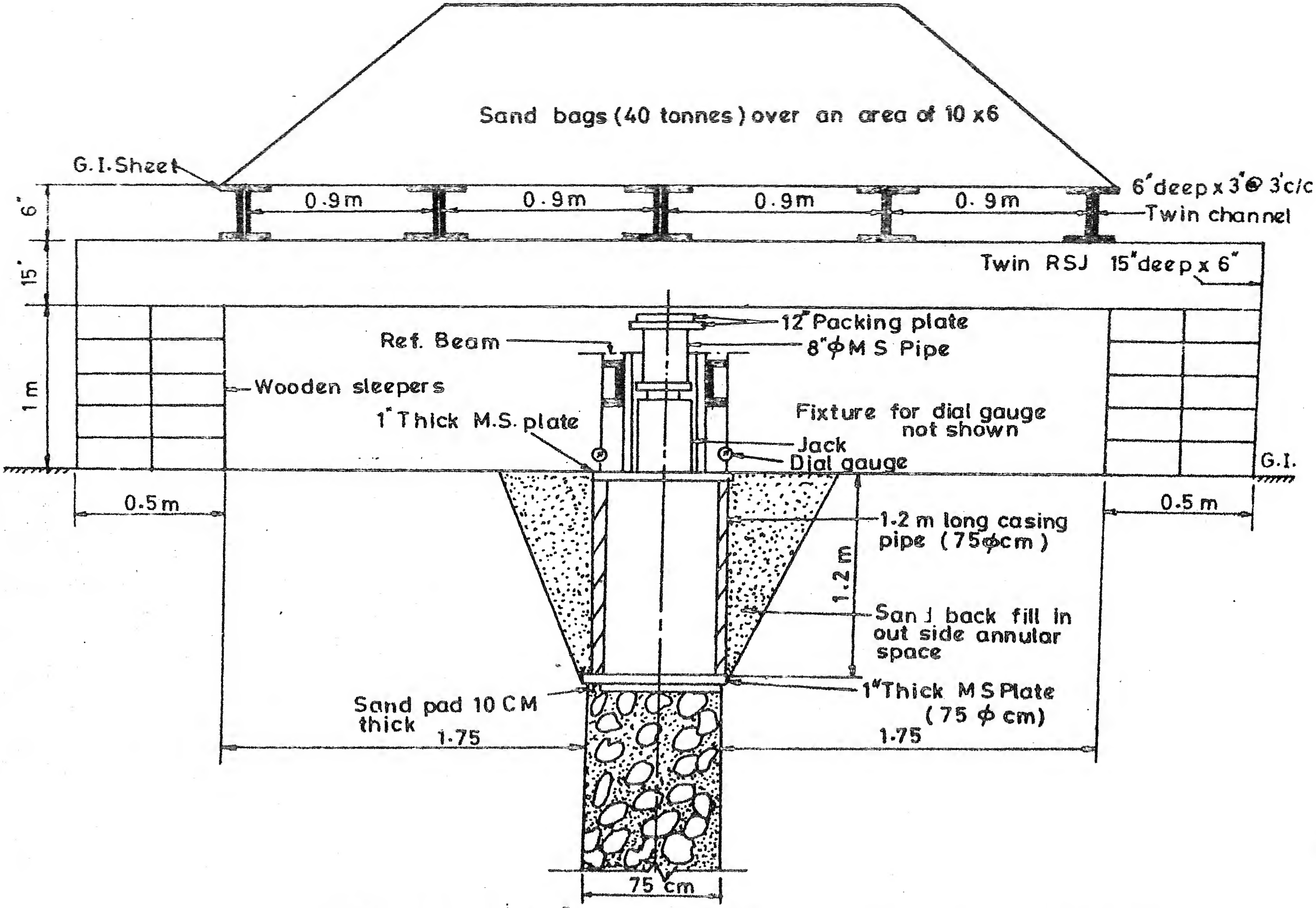


FIG. A-V ARRANGENT FOR SINGLE COLUMN LOAD TEST SET UP

REFERENCES

1. Adachi, K. and Todo, H. (1979), 'A case study on settlement of soft clay in Penang', 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering.

2. Basudhar, P.K. (1976), 'Some application of mathematical programming techniques to stability problems in geotechnical engineering'. A thesis submitted in partial fulfilment of the requirement for the degree of Ph.D. to the department of Civil Engineering, I.I.T.Kanpur.

3. Biot, M.A. (1941), 'General theory of three dimensional consolidation', Journal of applied physics, Vol.12, pp. 155-164.

4. Bishop, A.W. (1953), 'The use of slip circle in the stability analysis of earth slopes', Geotechnique,London, Vol.5, No.1, March, pp. 1-17.

5. Brown, J.D. and Meyerhof, G.G.(1969), 'Experimental study of bearing capacity in layered clays'. Proceedings of 7th International conference on Soil Mechanics and Foundation Engineering, Mexico, p.p. 45-51.

6. Burland, J.B. (1969), 'Deformation of soft clay beneath loaded areas', Proceedings, 7th International Conference on Soil Mechanics and Foundation Engineering, pp. 55-63.

7. Burland, J.B. (1971), 'A method of estimating the pore pressures and displacements beneath embankments on soft, natural clay deposits', stress-strain behaviour of soils, Edited by R.H.G. Parry pp. 505-536.

8. Button, S.J. (1953), 'Bearing capacity of footings on two layer cohesive subsoil', Proceedings of 3rd International Conference on Soil Mechanics and Foundation Engineering Zurich, Vol.1, p.p. 332-335.

9. Chen, W.F. (1975), 'Limit analysis and soil plasticity', Elsevier Publishing Co., New York.

10. Cox, A.D., Eason, G. and Hopkins, H.G.,(1961), 'Axially symmetric plastic deformation in soils', Transaction of Royal Society of London.

11. D'Appolonia, D.J., and Lambe, T.W. (1970), 'A New method for predicting initial settlement ', Journal of SMFE Division ASCE, Vol.46, No.SM2, March p.p. 523-545.

12. D'Appolonia,D.J., Poulos, H.G. and Ladd C.C. (1971), 'Initial settlement of structures on clay', Journal of SMFE Division ASCE, Vol. 97, No. SM10, October p.p.1359-1377.

13. Das, B.M. (1981), 'Bearing capacity of eccentrically loaded surface footing on sand', Soils and Foundation. The Japanese society of SMFE. Vol.21, March, No.1, p.p. 115-119.

14. Datye, K.R. and Nagraju, S.S. (1975), 'Installation and testing of rammed stone columns', Proceedings of speciality session of 5th Asian Regional Conference, Bangalore.

15. Datye, K.R. (1981), 'Low cost and energy saving techniques for ground improvement, soil stabilisation and soil reinforcement', Geomech 81, The fourth annual lecture of Indian Geotechnical society, Hyderabad p.p. 28.

16. Davis, E.H., and Raymond, G.P., (1965), 'A nonlinear theory of consolidation', Geotechnique, London, Vol.15, No.2, June pp. 161-173.

17. Davis, E.H., and Poulos, H.G. (1968), 'The use of elastic theory for settlement under three-dimensional conditions', Geotechnique, London, Vol.18, No.1, March p.p. 67-91.

18. Davis, E.H. (1971), 'Non-linear consolidation and the effect of layer depth', Research Report No. R168, School of Civil Engineering, The University of Sydney, Australia.

19. Davis, E.H. and Poulos, H.G. (1972), 'Rate of settlement under three dimensional conditions', Geotechnique, London, Vol.22, No.1, March, p.p. 95-114.

20. De Beer, E.E. (1970), 'Experimental determination of the shape factors and the bearing capacity factors of sand', Geotechnique, London, Vol.20, No.4, December, p.p.387-411.

21. Desai, C.S. and Able, J.F. (1972), 'Introduction to the finite element method', Affiliated East-west press pvt. ltd.

22. Desai, C.S., and Christian, J.T., (1977), 'Numerical methods in Geotechnical Engineering', McGraw-Hill Book Co.

23. Egorov, K.E., Kuzmin, P.G. and Popav, B.P.(1957), 'The observed settlements of buildings as compared with priliminary calculations', Proceedings of 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol.1, pp. 291-296.

24. Engelhardt, K., and Golding, H.C., (1975), 'Field testing to evaluate stone column performance in a seismic area', Geotechnique, London, Vol.25, No.1, March, p.p.61-69.

25. Foott, R. and Ladd, C.C. (1981), 'Undrained settlement of plastic and organic clays', Journal of Geotechnical Engineering Division ASCE, Vol.107, No. GT 8, August, pp. 1079-1094.

26. Gibson, R.E. and Lumb, P. (1953), 'Numerical solutions of some problems in consolidation of clay', Proceedings of the Institution of Engineers, London, Part I, January, p.p. 182-192.

27. Greenwood, D.A. (1970), 'Mechanical improvement of soils below ground surface', Ground Engineering, Proceedings of the conference organised by the Institution of Civil Engineers London, p.p. 16-18.

28. Harr, M.E., (1966), 'Foundations of theoretical soil mechanics', McGraw-Hill, New York.

29. Hughes, J.M.O., and Withers, N.J. (1974), 'Reinforcing of soft cohesive soil with stone columns', Ground Engineering May, p.p. 42-49.

30. Hughes, J.M.O., Withers, N.J., and Greenwood, D.A.(1975), 'A field trial of the reinforcing effect of a stone column in soil', Geotechnique, London, Vol.25, No.1, March, p.p. 31-34.

31. Ismeal, N.F., and Vesic A.S. (1981), 'Compressibility and bearing capacity', Journal of Geotechnical Engineering Division, ASCE, Vol.107, No. GT12, December pp.1677-1691.

32. Indian Standard Institution Code No. 1888, 'Method of load test on soils'.

33. Janbu, N. (1957), 'Earth pressures and bearing capacity calculations by generalised procedure of slices', Proceedings of 4th International Conference on Soil Mechanics and Foundation Engineering, London, Vol.12.

34. Kenney, T.C. (1964), 'Pore pressures and bearing capacity of layered clays', Journal of SMFE, Division, ASCE, Vol.90, No. SM4, July, p.p. 27-55.

35. Kezdi, A. (1961), 'The effect of inclined loads on the stability of a foundation', Proceedings of 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, Vol.I, p.p. 699-703.

36. Kezdi, A., Marczal, L., Jancsecz, S.,(1976), 'Settlement of tall, tower building subjected to time dependent load'. Proceedings of 5th Budapest Conference on S MFE, pp.327-336.

37. Kotzias, P.C. and Stamtopoulous, A.C. (1969), 'Preloading for heavy industrial installations', Journal of Soil Mechanics and Foundation Engineering ASCE, SM 6, November, pp.1335-1355.

38. Krishnamurty, S. (1972), 'Limiting equilibrium to a class of stability problems in soil mechanics', A thesis submitted in partial fulfilment of the requirement for the degree of Ph.D. to the department of Civil Engineering, I.I.T. Kanpur.

39. Lambe, T.W. and Whitman, R.V. (1976) , 'Soil Mechanics', Wiley Eastern Limited, New Delhi.

40. Lumb, P. (1965), 'Influence of construction time on pore pressure dissipation beneath foundations',Civil Engineering and Public Works Review, Vol.60,No.706,May,p.p.679-682.

41. Madhav, M.R. (1967), 'Clay Consolidation under construction loading with varing coefficient of consolidation', Journal of Indian National Society of Soil Mechanics and Foundation Engineering, Vol.6, No.2, April, p.p.145-152.

42. Madhav, M.R. and Vitkar, P.P.,(1978), 'Strip footing on weak clay stabilised with granular trench or pile', Canadian Geotechnical Journal, Vol.15, No.4, November.

43. Madhav, M.R. (1981), 'Gravel/sand pile reinforced soil', Annual report to Central Board of Irrigation and Power.

44. Madhav, M.R. and Vitkar, P.P. (1981), 'Responce of finite saturated layer to construction type of loading',International symposium on Mechanical Behaviour of structured media, Ottawa, May.

45. Matsuo, M., and Kuroda, K., (1974), 'Probablistic approach to design of embankments', Soilsand Foundation, Japanese Society of SMFE, Vol.14, No.2, June, p.p.1-17.

46. McKenna, W.A.E., and Wolstenholme, D.R.,(1975), 'Performance of an embankment supported by stone columns in soft ground', Geotechnique, London, Vol.25, No.1, March, p.p.51-59.

47. Meyerhof, G.G. (1957), 'The ultimate bearing capacity of foundations', Geotechnique, London, Vol.2, No.4, December, p.p. 301-332.

48. Meyerhof, G.G. (1953), 'The bearing capacity of foundations under eccentric and inclined loads', Proceedings of 3rd International conference on Soil Mechanics and Foundation Engineering, Zurich, Vol.I, p.p. 440-445.

49. Meyerhof, G.G. (1965), 'Some recent research on the bearing capacity of foundations', The Canadian Geotechnical Journal, Vol.I, No.1, September, p.p. 16-26.

50. Nakase, A. (1981), 'Bearing capacity of rectangular footings on clay of strength increasing linearly with depth', Soil and Foundations, The Japanese society of SMFE, Vol.21, No.4, December, pp. 101-108.

51. Poulos, H.G. and Davis, E.H. (1981), 'Pile foundation analysis and design', John Wiley and Sons.

52. Rao, B.G. and Bhandari, R.K. (1979), 'Reinforcing of noncohesive soils by granular piles', 6th Asian regional conference on Soil Mechanics and Foundation Engineering.

53. Raymond, G.P. (1967), 'Bearing capacity of large footings and embankments on clays', Geotechnique, London, Vol.17, No.1, March, p.p. 1-10.

54. Reddy, A.S., and Mogaliah, G. (1975), 'Bearing capacity of shallow foundations on slopes', Indian Geotechnical Journal, Vol.5, No.4, October, p.p. 237-253.

55. Reddy, A.S. and Rao, K.N.V.K. (1981), 'Bearing capacity of strip footing on anisotropic and nonhomogeneous clay', Soils and foundation , The Japanese Society of SMFE, Vol.21, No.1, March p.p. 1-6.

56. Road Research Laboratory (1968), 'Soil Mechanics for Road Engineers', Her majesty's stationary office ,eighth impression.

57. Schiffman, R.L. (1958), 'Consolidation of soil under time dependent loading and varying permeability', Proceedings of Highway Research Board, Vol.37, p.p. 584-617.

58. Schiffman, R.L., and Gibson, R.E., (1964), 'The Consolidation of nonhomogeneous clay layers', Journal of SMFE Division ASCE, Vol.90, No. SM5, September, p.p. 1-30.

59. Schnabel, J.J. (1972), 'Foundation construction on compacted structural fill in Washington D.C. Area', speciality conference on earth and earth supported structures Vol. I, part 2, p.p. 1019-1036.

60. Scott, R.F. (1963), 'Principles of Soil Mechanics', Addison-Wesley Publishing Co. Inc.

61. Skempton, A.W. and Bjerrum, L. (1957), 'A contribution to the settlement analysis of foundation in clay', Geotechnique, London.

62. Skempton, A.W. (1964),'Long term stability of clay slopes', Fourth Rankine lecture, Geotechnique, London, Vol.14, No.2, June, p.p. 77-102.

63. Simons, N.E. and Som, N.N. (1969), 'The influence of lateral stresses on the stress deformation characteristics of London clay', Proceedings of the 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, Vol.1, pp. 369-378.

64. Sokolovsky, V.V. (1960), 'Statics of soil media', 3rd edition, Fizmatgiz, Moscow.

65. Soneja, M.R. and Jain, P.K. (1981), 'Prediction of footing settlements in silts', Journal of Institution of Engineers (India), Vol.62, Part CI 3. November, p.p.176-181.

66. Taylor, T.W. and Ooi, T.A. (1971), 'The increase in bearing capacity resulting from consolidation', Geotechnique, London, Vol.21, No.4, December, p.p. 376-390.

67. Tejchman, A . (1977), 'Bearing capacity model test of stratified subsoil loaded by strip foundation', Canadian Geotechnical Journal, Vol.14, No.1, February,p.p.138-143.

68. Terzaghi, K. (1943), 'Theoretical Soil Mechanics', John Wiley and Sons, New York.

69. Tominaga, N., Sakaki, T., Hoshimoto, M., and Paulind, C.C.(1979),'Embankment tests in reclamation area,Manila Bay', 6th Asian Regional Conference on Soil Mechanics and Foundation Engineering.

70. Tomlinson, M.J. (1978), 'Foundation Design and Construction', Pitman Publishing Limited, London.

71. Tschebotarioff, G.P. (1951), 'Soil Mechanics, Foundations and Earth Structures', GcGraw Hill, New York.

72. Vesic, A.S., (1972), 'Expansion of cavities in infinite soil mass', Journal of SMFE Division, ASCE, Vol.98, No. SM3, March, pp. 265-290.

73. Vesic, A.S. (1973), 'Analysis of ultimate loads on shallow foundations', Journal of SMFE Division, ASCE, Vol.99, No. SM1, January , p.p. 45-73.

74. Vesic, A.S. (1975), 'Bearing capacity of shallow foundations', Foundation Engineering Handbook Edited by Winterkorn and Fang, Van Nostrand Reinhold Co., New York, p.p. 121-147.

75. Winterkorn, H.F. and Fang, H.Y. (1975), 'Foundation Engineering Handbook', Van Nostrand Reinhold Co. New York.

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